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Long-span Timber Floor Solutions

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1 Introduction

This guide covers two timber floor solutions – cassette type floors (using LVL or glulam web and LVL or CLT flanges) and panel-type floors (using CLT or combination of CLT with LVL or glulam secondary members) – that have the potential to be used for at least 9 x 9 metre mid-rise commercial building.

These floor alternatives have been arrived at, based on industry input, to address key concerns when designing long-span timber floors: constructability and floor dynamics.

Recommendations on design criteria, procedure and parameters for vibration design are based on existing knowledge from literature and supported by extensive laboratory tests.

It is well understood from previous studies that once the floor span exceeds 6 m, serviceability limit state requirements, especially vibration behaviour, rather than strength limit state requirements tend to govern the design. This guide addresses the performance requirements of the floors to meet the strength and serviceability limit state design requirements and the focus will be on design considerations for floor dynamics. The design process for the two floors is presented in two separate sections (Sections 3 and 4) but some of the steps and design criteria are common for both types.

These floors have been designed to be able to satisfy serviceability and ultimate limit state design as well as to ensure that both the systems are modular, suit prefabrication and are simple to assemble on site. The proposed panel-type floor can be built using CLT only or a combination of CLT supported on secondary LVL or glulam members while the cassette floors can be built into box-beam type sections but it may be beneficial to use the floor cavity for installing services and insulation. Access to the floor cavity in such case will require either the top or the bottom flange to be a non-structural component.

2 Floor Design Requirements

Performance requirements of a ribbed deck floor must address ultimate and serviceability limit states. Load type, load combinations and modification factors for both ultimate and serviceability limit states have been defined in accordance with the AS 1170 standards. The limit states that require checking, which have also been identified in previous studies on design requirements for long-span timber floors (WoodSolutions Technical Design Guide #31), are:

- Short-term ultimate limit state – response of the structure under maximum load.
- Long-term ultimate limit state – response of the structure to quasi-permanent loading and avoiding failure due to creep of the timber member in particular.
- Short-term serviceability limit state – instantaneous response of the structure to an imposed load.
- Long-term serviceability limit state – time-dependent variations of the material properties to identify the service life behaviour.
- Serviceability limit state – instantaneous response to an imposed load of 1.0 kN at mid-span as an indication of dynamic behaviour. This criterion alone is, however, not sufficient in satisfying vibration design. Further checks, particularly for spans greater than 6 m, are required and are detailed in the later sections.

Once the span of the floors exceeds 6 m, it is likely that the design will be governed by vibration and therefore more rigorous vibration design checks will be essential. This design guide will, therefore, focus on these additional design checks. Fire and acoustic designs are outside the scope of this design guide.

3 Design of Cassette Floor System

3.1 Overview

Ribbed-deck cassette floors consist of timber joists rigidly connected to a flange. Engineered wood products (EWPs) are used to make up the cassette where LVL or glulam can be used as the web while the flange can be made from LVL or CLT. The cassette can be manufactured from off-the-shelf products to reduce costs and streamline the fabrication process. Typical dimensions and grades for an LVL ribbed-deck cassette, shown in Table 1, are based on Nelson Pine LVL products. To achieve composite action, the flange should be connected to joists with a combination of adhesives (e.g. Purbond) and mechanical connectors (screws). Composite action is essential to increase stiffness and allow for longer spans.

From an ultimate limit state perspective, spans over 9 m are achievable, making it a suitable option for commercial buildings. However, the design is typically governed by vibration serviceability under human-induced walking excitations, which is a current area of investigation internationally and at the University of Technology, Sydney.

Several configurations of the web and the flange can be used e.g. flange connected to the top of the web only (Figure 1(a)), flange connected to the bottom of the web only (Figure 1(b)) or web sandwiched between both a top and bottom flange (Figure 1(c)). Each configuration has its own advantages and disadvantages in terms of construction and services installation and should be chosen based on the building requirements. This guide refers to a design procedure based on a ribbed-deck floor with top panel only. However, all the design criteria and considerations remain the same for other configurations.

Table 1: Typical dimensions of ribbed deck cassette floor based on Nelson Pine LVL products.

Component	Typical dimensions	Typical grade
Web	45 or 63 mm breadth x 300 or 360 or 400 mm depth	LVL13
Flange	45, 63 or 90 mm thickness x 1220 mm width	LVL11 or LVL13

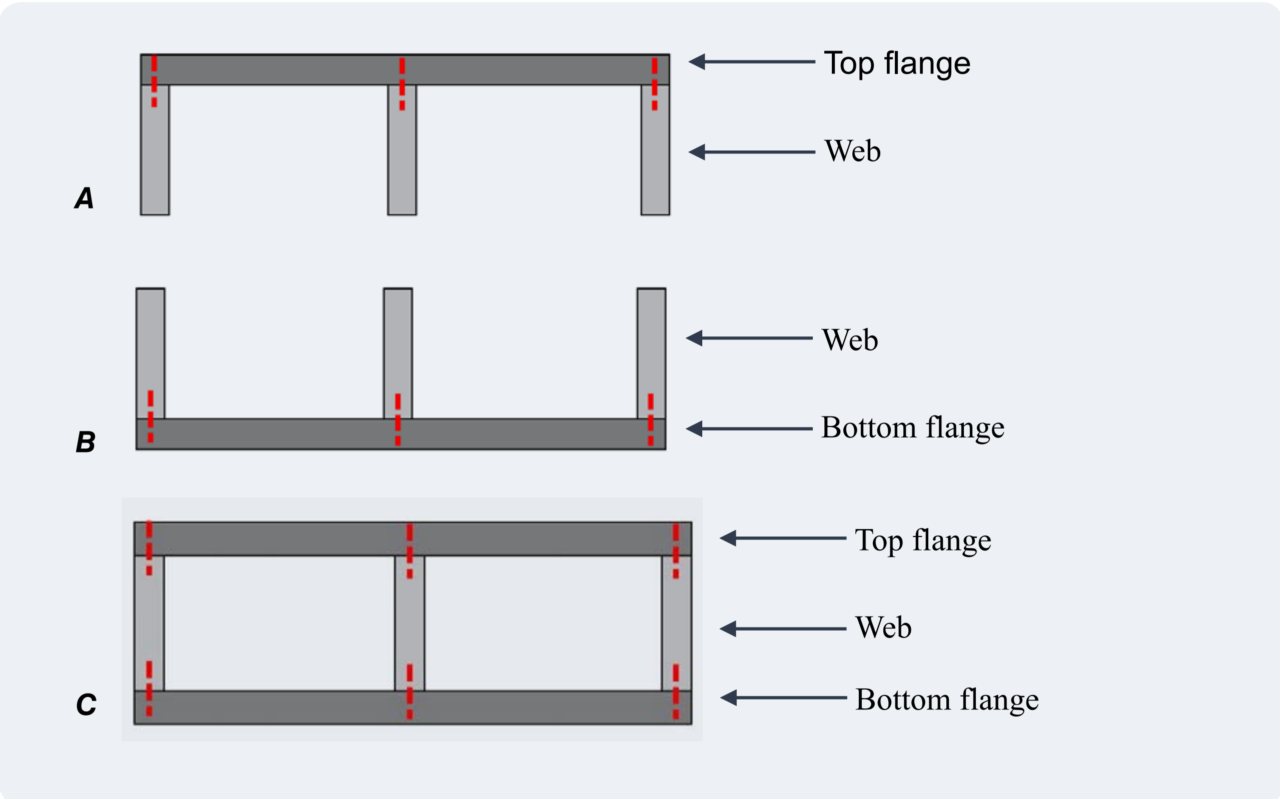


Figure 1: (a) top flange and web configuration; (b) bottom flange and web configuration; (c) web members sandwiched between top and bottom flange.

3.2 Design Considerations

3.2.1 Shear Deformation

Deflection of beams consists of bending and shear deformation. Typically, in concrete and steel, shear deformation in steel and concrete floor systems is negligible as most deflection is dominated by bending. However, timber has a low shear modulus, for example 0.66 × 10^3 MPa for LVL13 grade compared to 80 × 10^3 MPa for structural steel. As a result, the ratio of Modulus of Elasticity to shear modulus is high (i.e. 25 for LVL13), which can indicate that the shear component of total deflection becomes more significant (Skaggs and Bender 1995). Consequently, it is recommended that shear deformation is considered in both design and finite element modelling methods.

The American Institute of Steel Construction (AISC) Design Guide 11 (Murray et al. 2016) provides an equation for a reduced effective moment of inertia, I_e, for composite steel-concrete beams and trusses which accounts for shear deformation. It is recommended that this reduced effective moment of inertia be used for serviceability design:

$$I_e = \frac{I_{comp}}{1 + 0.15\, I_{comp}/I_{chords}}$$

where I_{comp} = fully composite transformed moment of inertia and I_{chords} = moment of inertia of chord or joist areas alone.

3.2.2 Shear Lag Effect

As the floor undergoes bending, there is a shear transfer between the web and flange members. However, Figure 2 shows that the stress distribution across the flange is not uniform. The shear lag effect considers this variation or 'lag' in stress via an effective flange width. This is defined as the width of panel, which effectively contributes to the stiffness of the floor system.

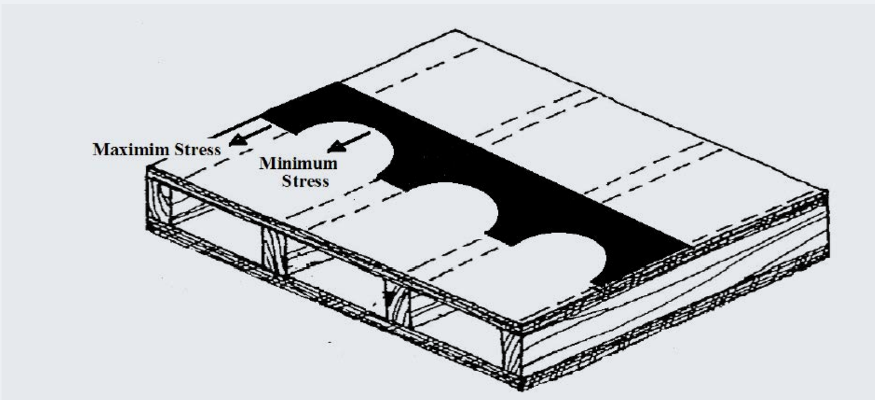


Figure 2: Shear lag effect in a ribbed deck floor system (Zabihi 2014).

Eurocode 5 (2004b) provides effective width calculations for shear lag effect in thin flanges such as plywood, oriented strand board and particleboard. The assembly is considered as a number of I-beams or U-beams (see Figure 3). Plate buckling effects are also considered for the compression flange (top flange), however, this is unlikely to occur in thick flange members used in long-span cassette floors.

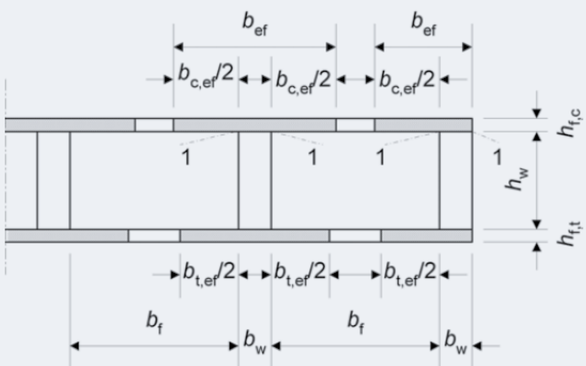


Figure 3: Thin-flanged beam (CEN 2004b).

Although there is no guidance for LVL or CLT flanges, which will be thicker than typical materials used in residential buildings such as plywood, OSB, etc, designers should ensure that the centre-to-centre web spacing is such that shear lag effects in the flange do not occur. If the effective width is greater than the web centre-to-centre spacing, shear lag effects will not occur. Based on Clause 9.1.2. Eurocode 5 (CEN 2004b), Equations 3.2 and 3.3 can be used to calculate the effective width of the top and bottom flanges. As an example, a 9000 mm spanning cassette with 90 mm top flange and 63 mm wide joist can have a maximum centre-to-centre spacing of 963 mm before shear lag effects occur in the flanges.

Bottom flange:

$$b_{f,t} = b_w + 0.1 \times span \tag{3.2}$$

Top flange:

$$b_{f,c} = \min(b_w + 0.1 \times span, b_w + 20 \times h_f) \tag{3.3}$$

where $b_{f,t}$ and $b_{f,c}$ are the effective width of the top bottom (tension) and top (compression) flange, respectively, and b_w and h_f is the web breadth and thickness of top flange, respectively. For $b_{f,c}$, the effective width considers plate buckling; however, this is unlikely to occur in thick flange members used in long-span cassette floors.

3.2.3 Blocking

Blocking or bridging provides lateral stability to the web which is particularly crucial for long-span cassette floors with no top flange. According to Ozelton and Baird (2006), lateral buckling of a beam depends on:

- depth-to-breadth ratio (or I_x/I_y ratio)
- the geometrical and physical properties of the beam section
- the nature of the applied loading with respect to the neutral axis of the section
- the degree of restraint provided at the vertical supports and at points along the span.

For ribbed deck floor systems, the top flange provides a degree of lateral restraint. Equation 3.4 from Clause 3.2.3.2 (b) in AS 1720.1 (2010) should be satisfied in order to determine whether the top flange provides continuous lateral restraint to the web member. L_{ay} refers to the distance between points of effectively rigid restraints against lateral movement i.e. distance between screws connecting the flange to web. If this equation is satisfied, blocking is not structurally required for ultimate limit strength, however, should still be used for ease of fabrication. For cassettes with only a bottom flange, blocking will be required to satisfy lateral buckling checks as per Clause 3.2.3 in AS 1720.1 (Standards Australia 2010).

$$\frac{L_{ay}}{d_w} \leq 64\left(\frac{b_w}{\rho_b d_w}\right)^2 \tag{3.4}$$

where d_w is the depth of the web and ρ_b is the material constant for a beam (Clause 3.2.4 AS 1720.1 (Standards Australia 2010).

3.2.4 Thinner Outer Joists

Outer joists for ribbed deck floor cassettes can be designed to have a thinner breadth since they only take half the load. This will create a more structurally efficient cross-section and save costs on material.

3.3 Design Procedure

The design procedure has following three fundamental stages:

1. Identifying the characteristics of the ribbed deck cross-section
2. Evaluation of the strength capacity
3. Assessment of the serviceability limit.

3.3.1 Cross-section Characteristics

Section properties of ribbed deck floor sections can be calculated assuming fully composite action between the web and the flange. Transformed section method can be used when the flange and the web have differing Modulus of Elasticity. Figure 4 shows notations for a typical cross-section of a ribbed deck floor with top and bottom flange.

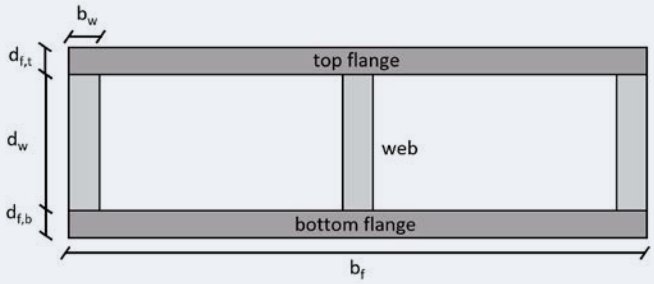


Figure 4: Notation for typical cross-section of ribbed deck floor system with top and bottom flange.

3.3.2 Evaluation of Strength Capacity

The following capacity checks need to be made:

- bending capacity of the cross-section
- axial capacity (compression) of the top flange
- axial capacity (tension) of the bottom flange (if present)
- combined bending and compression of the top flange
- combined bending and tension of the bottom flange (if present)
- shear capacity in web
- shear flow at interface between web and flanges
- shear strength at glue line
- bearing strength.

Appropriate equations of these checks are given in WoodSolutions Technical Design Guide #31: Timber Cassette Floors and can similarly be used for the design of ribbed deck cassette floors. The design procedure is based on AS 1730.1:2010 Timber structures Part 1: Design methods with load actions predicted using the AS 1170 Structural Design Action series.

3.3.3 Serviceability

Deflection

Deflection must be checked for short-term and long-term serviceability load combinations. The limits depend on the functional requirements of the building being designed. Appropriate equations for serviceability checks can be found in Wood Solutions Technical Design Guide #31: Timber Cassette Floors and can similarly be used for the design of ribbed deck cassette floors. A reduced effective moment of inertia is to be considered in deflection calculations as per Equation 1.

Vibration

Two approaches exist for the vibration design of floors: simplified design using hand calculations or a spreadsheet and finite element (FE) modelling. Choice of the method primarily depends on the complexity of the floor but is also influenced by the stage of structural design and the end-use of the floor. The orthotropic nature of timber cassette floors can result in closely spaced modes that can amplify the motion, negatively affecting the dynamic response of the floor (Khokhar, 2004). People tend to be more annoyed when there are two closely spaced frequencies (Ljunggren 2006; Ljunggren, Wang & Ågren 2007) and consideration of these higher modes during design is recommended (Brownjohn & Middleton 2008; Ljunggren 2006). Consequently, finite element modelling is recommended to obtain modal properties including frequency and mode shapes. Response analysis and assessment can then be performed, either through hand calculations using classic dynamic theory or through the FE model if the software allows. The following sections outline a vibration design procedure for long-span ribbed-deck cassette floors that stems from literature review, observations from testing at UTS on a 9 m span ribbed-deck floor (with top flange only) and current vibration design guides for other floor materials. Only dynamic loading from walking excitation is considered as it is the most common form of human induced excitation. The final project report, PNA 341-1415, for this FWPA-funded research is available on the FWPA website.

An overview of the stages of vibration design and associated questions is presented in Figure 5 and Table 2, respectively. The answers to the questions in Table 2 are primarily influenced by the floor use. Design processes for vibration are detailed in Section 4.5.

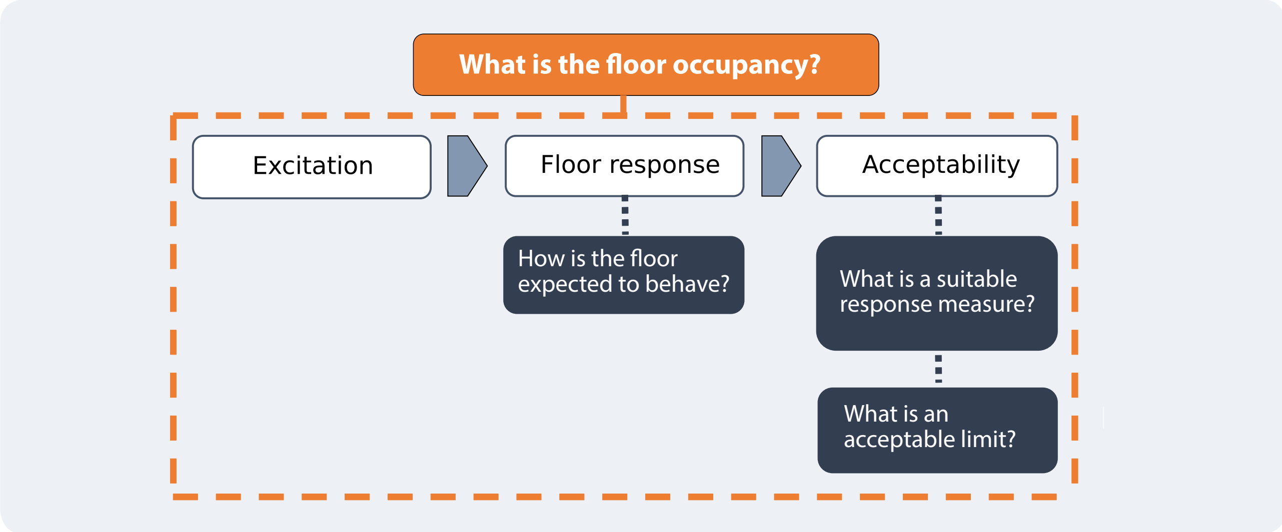


Figure 5: Flowchart of vibration assessment.

Table 2: Associated questions in reference to vibration assessment flowchart.

Question	Description
What is the floor occupancy?	Categorise the occupancy of the floor. This will indicate dead and live load requirements and the demands regarding floor vibration.
How is the floor expected to behave when subject to dynamic forces?	Determine modal properties of the floor using the simplified method or finite element modelling. Boundary conditions, loading, material properties, connection to adjacent cassettes and the main structure will influence the modal properties.
What limitation measure will you use?	Vibration characteristics include frequency, deflection, velocity or acceleration. Response factor is a commonly used measure.
What is an acceptable vibration limit for your floor?	This directly relates to the floor occupancy. High-importance buildings, such as hospitals and laboratories, will have more stringent vibration limits than a residential building.

3.4 Vibration Design Considerations for Ribbed Deck Floors

3.4.1 Modal Separation and Participation

It is apparent from experiments at UTS (final project report, PNA 341-1415) that the frequencies of the first bending and torsion modes of the floor are close together. Figure 6 shows the measured mode shapes, frequencies and damping ratios up to 40 Hz for a single cassette floor with overhanging flange bearing onto a rigid timber frame. Closely spaced modes are often caused by orthotropic floor systems in which the flexural rigidity along joists is higher than that across-joists and can amplify floor motions (Khokhar 2004). Further, people tend to be more annoyed when there are two closely spaced frequencies (Ljunggren 2006; Ljunggren, Wang & Ågren 2007). Although there is no clear definition of spacing required between modes to avoid interaction, a minimum separation of 5 Hz has been suggested (Ohlsson 1982; Weckendorf & Smith 2012).

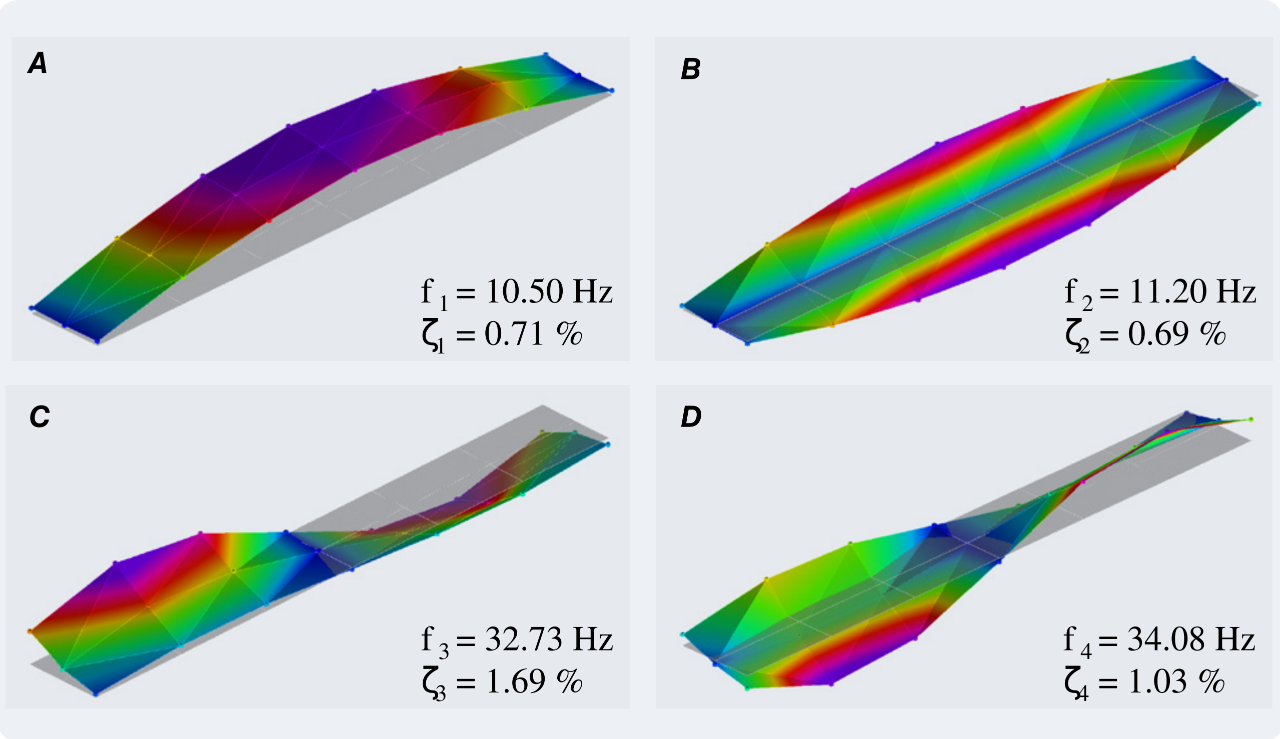


Figure 6: Extracted mode shapes for (a) Mode 1 – first bending mode (b) Mode 2 – first torsion mode (c) Mode 3 – second bending mode and (d) Mode 4 – second torsion mode.

3.4.2 Damping Ratio

Damping is the structure's ability to dissipate vibration energy through friction. It is made up of two main categories: material and structural damping. Material damping involves the internal friction within the material that is created through energy dissipation associated with microstructure defects such as grain boundaries and impurities (Labonnote 2012). Structural damping is a form of mechanical energy dissipation by friction of movement between components such as at support connections. Structural damping also depends on the occupancy, where friction between the floor and partitions or furniture can also contribute.

For timber floor systems, the main contribution to damping will be from structural damping. However, predicting damping is complex as slight differences in superimposed load and connection systems can affect the value. In addition, workmanship will also differ from site to site. The damping ratio also differs between modes and varies with load amplitude. For example, field testing of 13 different timber joist floors found that the average damping ratio for impulse loads was 5.05% as opposed to 0.95% for ambient vibration (Xiong, Kang & Lu 2011). For these reasons, standards often suggest a conservative value.

Damping values for long-span ribbed deck floors for commercial applications is currently limited. Testing on both single cassette and two adjacent connected 9 m spanning cassettes has shown that the damping ratio for the first and second bending mode is about 1%. Interestingly, under a simply-supported case, the damping ratio increased to 3–4% when a subject walked across the floor at various pace frequencies. Other studies on light-weight floor structures have also highlighted the positive influence of human-structure interaction on the damping ratio (Sachse 2002; Živanović, Diaz & Pavic 2009). This highlights the potential benefit of considering human-structure interaction in design approaches.

In current standards, Eurocode 5 suggests a damping ratio of 1% should be assumed 'unless other values are proven to be more appropriate' (CEN 2004b). This value is based on investigations undertaken by Ohlsson (1988b, 1991) on residential timber joist floors, which are typically constructed from solid timber joists nailed to thin top flanges made from oriented strand board (OSB) or plywood. More recent investigations have been carried out by Weckendorf et al. (2008) on composite LVL joists glued and screwed to an OSB decking among other configurations. Damping ratios of 2.0–3.5% were measured for the first mode while second and third modes had a mean value of about 1%. At this stage, it is recommended that a value of 1% is assumed for design. Table 3 compares the various damping ratios found in literature for more conventional timber joist floors and the influence of a screed layer. Concrete and steel floors have also been included for reference.

Table 3: Damping ratios for various floor systems.

Code/guideline	Floor type	Damping ratio
CCIP-016 (Willford & Young 2006)	Bare reinforced concrete floors	1%–2%
	Completed reinforced concrete floors with typical fit out	2.2–3.5%
	Completed steel composite, post tensioned or reinforced concrete floors with extensive fit out and full height partitions	3% 4.5%
Eurocode 5 (CEN 2004c)	Timber floors[1]	1%
UK NA to Eurocode 5 (BSI 2008b)	Timber floors	2%
ISO 10137:2007 (ISO 2007)	Wood joist floors – preliminary design value	2%
	Wood joist floors – typical range	1.5–4%
	Wood joist floors – extreme range	1.0–5.5%
HIVOSS (HIVOSS 2008)	Bare wood floor[2]	6%
Mohr (Mohr 1999)	Timber floors without any additional boardings for sound insulation	1%
	Plain glulam timber floors with additional boarding for sound insulation	2%
	Girder floors and nail laminated timber floors with additional boarding for sound insulation	3%
Hamm et al. (2010)	Timber floors without any floor finish	1%
	Plain glued laminated timber floors with floating screed	2%
	Girder floors and nail laminated timber floors with floating screed	3%

Notes:

[1] Unless other values are proven to be more appropriate.

[2] For open plan office, it is suggested to add another 1% damping due to the furniture.

3.4.3 Load Case

Overestimating the mass can be non-conservative for footfall vibrations (Willford & Young 2006). Floor loading should include the unfactored self-weight of the structure plus any superimposed dead load, such as any floor finishes and ceiling and services. Due to the requirement for commercial spaces to be flexible for different layouts and occupancies, it is recommended that about 10% of nominated live load is considered for vibration design (Smith, Hicks & Devine 2009). Mass for floor vibration can be calculated using Equation 3.5, where G, SDL and Q represent the self-weight, superimposed dead load and live load, respectively.

$$m = G + SDL + 0.1Q \tag{3.5}$$

3.4.4 Boundary Conditions

Boundary conditions significantly influence the modal properties of floor systems. For footfall analysis of steel and concrete floors using finite element models, connections between floor system and main structure are assumed to act as fixed due to the very small strains associated with footfall loading. However, there has been limited research into how accurate this assumption is for timber structures.

Ribbed deck floors will typically be connected into a timber frame system spanning between primary beams (see Figure 7(a)) or clamped between CLT walls (see Figure 7(b)). The clamping effect of the top flange will result in an increased rotational stiffness. However, the effect may be limited due to the compressibility of wood, which allows for rotational movement of the floor in the joints between walls and floor (Jarnero 2014). Impact hammer tests on a 9 m spanning ribbed deck cassette have shown that a flange-bearing support condition similar to Figure 7(a) acted very closely to a pin-support. Addition of an added load of up to 2000 N at each support location showed minimal increase in natural frequency for the first bending and torsion modes.

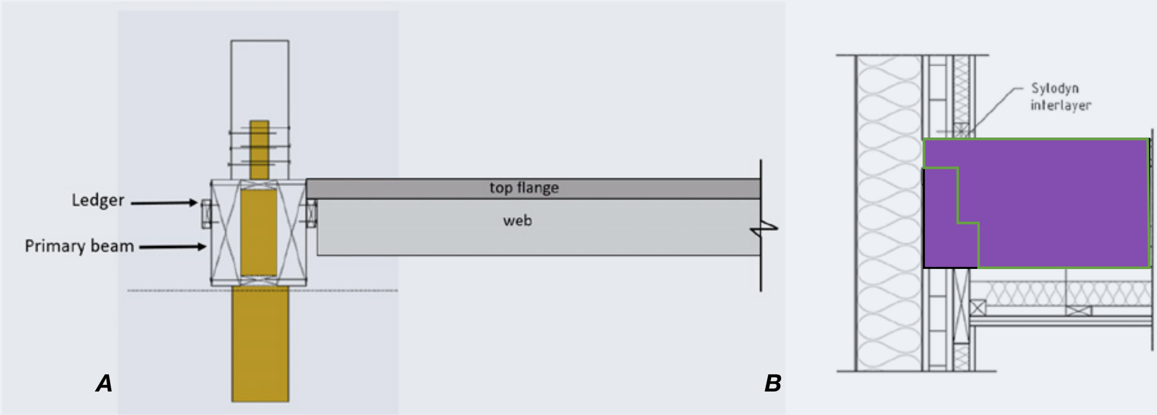


Figure 7: Example of connection of ribbed deck floor cassette into main structure (a) supported by primary beams, adapted from WoodSolutions Technical Design Guide #26 (Forsythe 2015) (b) supported on CLT load-bearing walls (Jarnerö, Brandt & Olsson 2015).

Another support condition often found in timber frame buildings in Europe involve the addition of an acoustic interlayer at boundaries between floor and supporting beam/wall, as shown as the 'Sylodyn interlayer' in Figure 7(b). Jarnerö, Brandt and Olsson (2015) investigated the effect of acoustic interlayers (Sylomer® and Sylodyn® manufactured by Getzner) on the damping ratio of a cross-laminated timber ribbed deck floor both in a laboratory environment and in-situ. Sylomer has a combination of both spring and damper properties while Sylodyn has stronger spring and smaller damping properties. Both elastomers are suitable for a wide range of applications including as a vibration isolation element in the rail industry, elastic machine bearing and to minimise footfall noise in buildings including those made from mass timber (Getzner GmbH 2016). Test results showed that the damping ratio increased to 6% when the last storey was added (floor tested was on second floor of an 8-storey building). This was significantly different to the 2.5% value obtained from the laboratory for the same elastic interlayer boundary condition.

Experiments on the influence of the Sylomer® SR 55 and Sylodyn® NB interlayer on the damping ratio of a long-span ribbed deck floor system were also undertaken at UTS. The ratio of utilisation was 70% and 96% for the Sylodyn and Sylomer interlayer, respectively; this means that the elastomers were still within the static range of operation. The elastomers were placed underneath an overhanging top flange bearing onto a timber frame support which was secured to the ground. Figure 8 shows the measured damping ratio for the first bending and torsion modes with and without the interlayer. Results show that for the Sylomer® interlayer, the damping ratio for modes 1 and 2 increased by over six- and nine-fold, respectively, compared to the case with no interlayer. Under walking tests at a pace frequency equivalent to the fifth integer of the fundamental frequency, the response factor at the centre of the floor reduced by 61% and 78% for Walker 1 and 2, respectively. It is important to note that the addition of the elastomer also introduced new modes under 50 Hz. Further tests would be needed to verify these results and whether any formal recommendations can be made on the use of an elastomer to mitigate floor vibration.

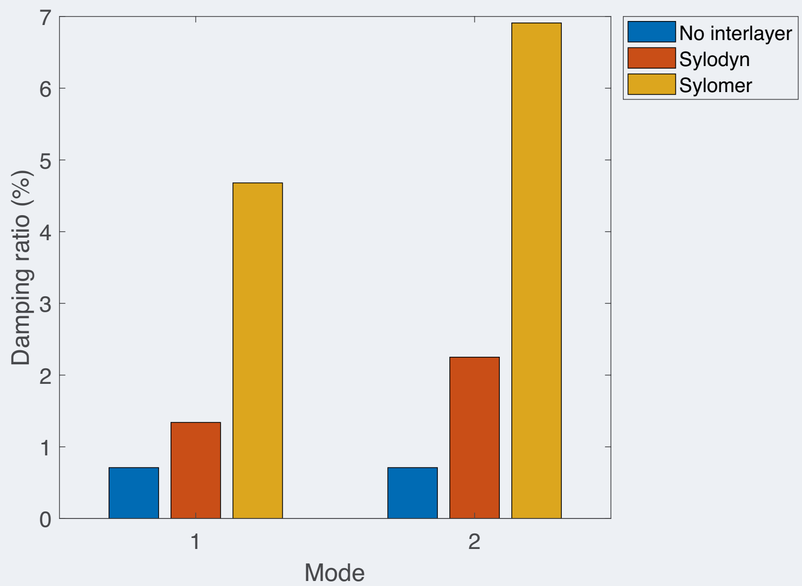


Figure 8: Effect of different interlayers on damping ratio of first bending and torsion modes.

3.4.5 Architectural Considerations

Dynamic behaviour of long-span ribbed deck floor systems will greatly benefit from cooperation between architects and structural engineers at an early design stage. If the footfall loading is close to the maximum deflection points in the mode shape, it is likely that there will be a higher floor response. Walking paths or corridors can be strategically placed closer to beams and columns as these areas will be less responsive than areas in the middle of the floor. Corridor length should also be considered as the longer the corridor, the more time is associated with walking (Smith, Hicks & Devine 2009).

3.5 Vibration Design Procedure

The following design procedure for a floor under footfall excitation is common to a number of vibration design guides, including CCIP-016(Willford & Young 2006), SCI P354 (Smith, Hicks & Devine 2009) and AISC DG 11 (Murray et al. 2016):

Step 1. Calculation of modal properties:

- a. Natural frequency
- b. Modal mass
- c. Mode shape

Step 2. Categorisation as a low or high frequency floor.

Step 3. Evaluation of response.

Step 4. Checking response against acceptance criteria.

The following sections provide commentary from current literature and equations, where available, on each step with reference to ribbed deck floor systems. A design for a simply-supported cassette within a commercial building is shown in Appendix A.

3.5.1 Step 1: Calculation of Modal Properties

As mentioned in Section 3.4.1, timber cassette floors are prone to having closely spaced modes with the first longitudinal bending mode not always being the most critical. Step 1 is recommended to be undertaken using a finite element model. Hand calculations using closed-form solutions for the fundamental mode can be used as a guide as to the potential nature of floor response.

Several assumptions are made when using an idealised situation (Bishop & Johnson 1960):

1. The system can be isolated from its surroundings. It is supported by rigidly-fixed points and are not affected by external forces. This ideal boundary does not exist in the real world.
2. The materials used to make up the system are perfectly homogenous and dimensions are exact. Wood, in particular, is a cellular material and is non-homogenous due to its anisotropic nature. Although engineered wood provides the user with a more reliable product, uncertainties still remain.
3. Some systems are assumed to have finite freedom only, they must be constructed of rigid bodies and massless springs.

Natural frequency

Ribbed deck floors have a similar structure to steel-concrete composite floor systems in which there are secondary beams or joists which are compositely connected to a flange element. With the floor cassettes most likely being supported by primary perimeter beams, consideration should be taken of the primary beam mode shapes about the columns. SCI P354 (Smith, Hicks & Devine 2009) suggests calculating the natural frequency of both secondary beam (Figure 9(a)), and primary beam modes (Figure 9(b)), and choosing the lower value. Mode A assumes that the ribbed deck cassettes vibrate as simply-supported members about the primary beams while Mode B assumes that the primary beams vibrate about the columns as simply-supported members and the ribbed deck cassettes are taken as fixed-ended.

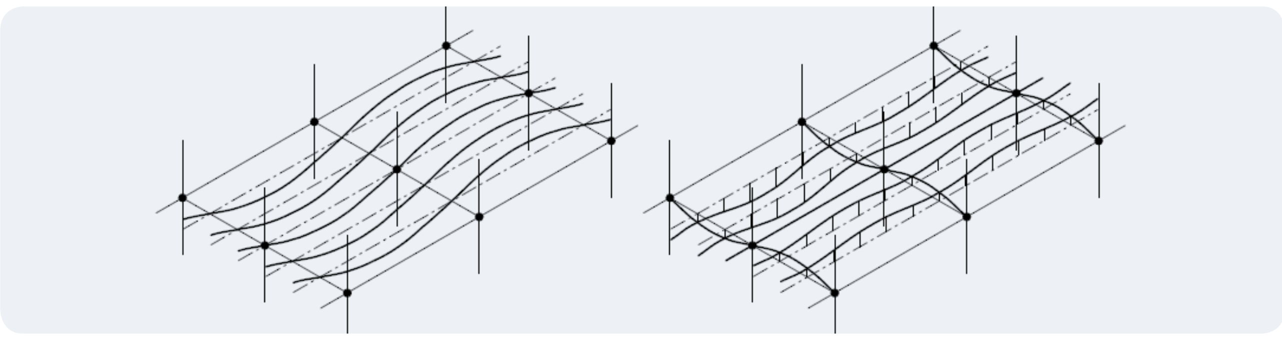


Figure 9: Mode shape governed by (a) secondary beam flexibility (b) primary beam flexibility (Smith, Hicks & Devine 2009).

The fundamental frequency can be calculated using the following equation:

$$f_0 = \frac{18}{\sqrt{\delta}} \qquad (3.6)$$

Where δ is the total deflection (in millimetres) of the ribbed cassette and primary beams, depending on the mode shape being considered. For Mode A, only the deflection of the ribbed deck floor needs to be considered. For Mode B, the deflection of the primary beam needs to be added to the deflection of the ribbed deck floor. If the primary beam dimensions and span are unknown, assessment of Mode A is sufficient. Equation 3.6 is a rearrangement of the equation for free elastic vibration of a simply-supported beam of uniform cross-section:

$$f_n = \frac{\pi^2}{2\pi}\sqrt{\frac{EI_e}{mL^4}} \qquad (3.7)$$

Deflection of a simply supported and fixed-fixed element subjected to a uniformly distributed load is shown in Equations 3.8 and 3.9.

Simply-supported case:

$$\delta = \frac{5mgL^4}{384EI} \qquad (3.8)$$

Fixed-fixed case:

$$\delta = \frac{mgL^4}{384EI} \qquad (3.9)$$

where:

E	is the static Young's modulus [N/mm^2]
I_e	is the effective second moment of area of one cassette as a composite section with consideration of shear deformation [mm^4]. For Mode A, if flange and web members are of different grades, the transformed section method can be used to calculate the neutral axis of the cassette cross-section.
m	is the mass per unit length as per Equation 3.9
L	is the length of the cassette [mm]

Modal mass

The modal mass is the amount of mass involved in the mode shape i.e. how much kinetic energy exists within the system. For the purpose of a simplified calculation, the modal mass of a simply-supported ribbed deck cassette can be as assumed to be similar to that of a simply-supported beam of uniform cross-section.

The modal mass can then be calculated as:

$$\hat{m} = \frac{mL}{2} \qquad (3.10)$$

Mode shape factor

The mode shape is the structure's preferred maximum displacement pattern when excited by a sudden impact and differ for each mode. The mode shape for the *j*th mode is:

$$\mu_j = sin\left(\frac{j\pi y}{L}\right) \qquad (3.11)$$

where, *y* is the distance along the beam of the excitation or response point.

The mode shape factor can be conservatively taken as 1, since the worst case for both the excitation and response point will occur at maximum deflection points in the mode shape. For first bending mode, *y/L*=0.5, and for second bending mode, *y/L*=0.25 and so on.

3.5.2 Step 2: Categorisation as a low- or high-frequency floor

Many guides, including CCIP-016, SCI P354 and DG11, categorise the floor response based on the fundamental frequency as either resonant or transient and suggest a different human loading force model for each case. Low-frequency floors are assumed to sustain resonance with a higher harmonic of the walking frequency with amplitudes building up as each footstep is taken. Guides suggest conservatively that the walking force is continuous and perfectly period and thus can be represented as a Fourier series of harmonic force contributions:

$$F(t) = Q + \sum_{h=1}^{H} \alpha_h Q \sin(2\pi h f_p t - \phi_h) \tag{3.12}$$

where $F(t)$ = vertical walking force; Q = static weight of an 'average' person (normally 76 kg × 9.81 m/s^2 = 746 N); h = harmonic number; n = total number of contributing harmonics; α_h = Fourier coefficient of the h-th harmonic generally known as the Dynamic Loading Factor (DLF); f_p = pace frequency (Hz); and Φ_h = phase lag for the h-th harmonic. CCIP-016, SCI P354 and DG 11 all suggest different DLFs based on various literature.

On the other hand, a high-frequency floor response is assumed to be characterised by an initial peak associated with each heel drop and decaying vibrations at a rate depending on the damping ratio. For this case, an impulsive footfall force model representing the heel strike is suggested. All guides have agreed on the method proposed by Willford et al. (2006) in which the initial and hence maximum velocity under an impulsive action can be calculated by dividing the magnitude by the modal mass. For unit mass, the initial velocity is numerically equal to the applied impulse, referred to as the 'effective impulse' and expressed as:

$$I_{eff} = \frac{f_p^{1.43}}{f_n^{1.3}} \frac{Q}{17.8} \tag{3.13}$$

CCIP-016 suggests a 'design' value of Equation 3.13 that has a 25% chance of being exceeded while SCI P354 incorporates requirements provided in EN 1990 Annex C (Gulvanessian 2001; Smith, Hicks & Devine 2009) that results in an 18% larger effective impulse than CCIP-016. Higher natural frequencies and low pace frequencies result in a lower effective impulse (Willford, Young & Field 2006). Guides only consider the fastest pace frequency expected in the occupancy, suggested as 2.5 Hz in CCIP-016 and 2.2 Hz for SCI P354 and DG11.

Despite this distinction, there have been many observed cases where 'high-frequency' floors have exhibited a resonant response or 'low-frequency' floors have localised high frequency modes with low modal mass which are easily excited by footstep impulses (Brownjohn, Racic & Chen 2016).

For long-span timber floors with low modal mass and first modal frequency in the 8 to 12 Hz range (depending on the loading considered), such a finding may be particularly relevant.

Walking tests have shown that although the fundamental frequency would classify the floor as a high-frequency floor, a resonant response was generated. Interestingly, contrary to the common assumption that only natural frequencies up to fourth harmonic should be considered, the resonant response was generated from a pace frequency in which the fifth harmonic was coinciding with the fundamental mode. This may indicate that the classification of long-span ribbed-deck floors as low- or high-frequency may not be appropriate. In addition, having two closely spaced modes around the cut-off frequency may create further uncertainty of this approach.

Adding to the ambiguity of the categorisation, various guidelines and standards recommend different cut-offs (see Table 4). For current designs, as per SCI P354, it is suggested that low-frequency floors are checked for both resonant and transient responses while high-frequency floors are checked for only transient response.

Table 4: Low to high frequency floor cut-off for various guidelines and standards.

Reference	Low to high frequency cut-off
SCI P354 (Smith, Hicks & Devine 2009)	10 Hz
Concrete Centre (Willford & Young 2006)	10.5 Hz[1]
Toratti and Talja (2006)	10 Hz
BS 6472-1:2008 (BSI 2008a)	7–10 Hz
Allen and Murray[2] (1993)	9 Hz
Wyatt and Dier[2] (1989)	7 Hz
Ohlsson[2] (1988a)	8 Hz

Notes:

[1] 4.2×maximum footfall rate (2.5Hz)

[2] As per Pavic et al. (2003)

3.5.3 Step 3: Evaluation of Response

Apart from the natural frequency, deflection, velocity and acceleration can also be used to quantify the response of the floor. These parameters stem from the dynamic equilibrium equation for a single-degree-of-freedom system subjected to an external dynamic force, *p(t)*:

$$m\ddot{y}(t) + c\dot{y}(t) + ky(t) = p(t) \tag{3.14}$$

where, *m, c* and *k* are the matrices of mass, damping and stiffness, respectively and $\ddot{y}(t)$, $\ddot{y}(t)$ and $y(t)$ are acceleration, velocity and deflection, respectively. Acceleration is the most commonly used evaluation parameter as easily correlated to measurements from accelerometers and also appears to be the best parameter to relate to acceptable magnitudes of human perception of motion (Irwin 1978). Depending on the guide, peak or root-mean-square (RMS) acceleration are typically suggested as an evaluation for both resonant and transient response floors. Table 5 shows the different evaluation parameters suggested by other guides and the subsequent final evaluation criterion. Note that the RMS acceleration of a sine wave is about 70% of the peak acceleration (Equation 3.15).

$$a_{rms} = \frac{a_{peak}}{\sqrt{2}} \tag{3.15}$$

Table 5: Response evaluation for CCIP-016, SCI P354 and DG 11.

Guideline	Resonant response parameter	Transient response parameter	Final assessment
CCIP-016	Peak acceleration	RMS velocity	Response Factor
SCI P354	RMS acceleration	RMS acceleration	Response Factor and/or Vibration Dose Value
DG 11	Peak acceleration	Equivalent sinusoidal peak acceleration	%g

Resonant response

Resonant response analysis involves the calculating the total response of each harmonic of walking which is found through the square-root sum of squares of the acceleration response of each relevant vibration mode of the system. Typically, the first four harmonics are considered while all modes up to 15 Hz for CCIP-016 and 12 Hz for SCI P354 are included. The general expression for total acceleration response at a position *r* from excitation at a point *e* is shown in Equation 3.16 taken from SCI P354; note that CCIP-016 and DG 11 expressions will vary slightly to the equation.

$$a_{w,rms,e,r} = \frac{1}{\sqrt{2}}\sqrt{\sum_{h=1}^{H}\left(\sum_{n=1}^{N}\left(\mu_{e,n}\mu_{r,n}\frac{F_h}{M_n}D_{n,h}W_h\right)\right)} \tag{3.16}$$

where H = number of harmonics; N = number of modes; h = harmonic number; n = mode number; $\mu_{e,n}$ = mode shape amplitude at the point on the floor where excitation force is applied; $\mu_{r,n}$ = mode shape amplitude at the point where response is calculated; F_h = excitation force for the h-th harmonic ($F_h = \alpha_h Q$); M_n = modal mass of mode n; $D_{n,h}$ = dynamic magnification factor for acceleration; W_h = appropriate code-defined weighting factor for human perception of vibrations for the frequency of the harmonic under consideration h_{fp}. The worst case for the mode shape amplitudes is when the excitation and response locations are at the same point. When using a finite element model for vibration design, response at each node for excitation at each node should be checked to obtain the worst case. When using hand calculation methods, mode shape amplitudes can conservatively be taken as 1.

The dynamic magnification factor for acceleration is the ratio of the peak amplitude to the static amplitude and can be calculated as follows where β_n is the frequency ratio of f_p/f_n:

$$D_{n,h} = \frac{h^2\beta_n^2}{\sqrt{(1-h^2\beta_n^2)^2 + (2h\zeta\beta_n)^2}} \tag{3.17}$$

A resonance build-up factor, shown in Equation 3.18, can also be applied for each harmonic at each relevant mode; this factor, related to the damping and the number of footsteps taken to cross the span, reduces the extent of full resonant build-up. Although, since architectural layout of corridors and partitions may not be known, it is commonly taken as 1.

$$\rho_{h,n} = 1 - e^{-2\pi\zeta N} \quad \text{where} \quad N = 0.55h\frac{L}{l} \tag{3.18}$$

where ζ = damping ratio; N = number of footsteps; L = span; l = stride length (typically 0.75 m for 2 Hz walking pace).

Transient response
Transient response analysis typically involves all modes up to twice the fundamental frequency. Since faster walking speeds generally induce a higher response, only the fastest walking pace that is expected on the floor is considered; for corridors and circulation zones, this is typically 2.5 Hz (Willford & Young 2006). The acceleration of each impulse is typically expressed as the sum of responses for each relevant mode:

$$a_{w,e,r}(t) = \sum_{n=1}^{N} 2\pi f_d \mu_{e,n}\mu_{r,n}\frac{I_{eff}}{M_n}\sin(2\pi f_d t)\cdot e^{-2\pi\zeta f_n t}W_n \tag{3.19}$$

where f_d = damped natural frequency $f_d = f_n\sqrt{1-\zeta^2}$; W_n = appropriate code-defined weighting factor for human perception of vibrations for the frequency of the mode under consideration f_n. Equation 3.20 can then be used to determine the RMS acceleration where $T = 1/f_p$.

$$a_{w,e,r} = \sqrt{\frac{1}{T}\int_0^T a_{w,e,r(t)}dt} \tag{3.20}$$

Response factor
CCIP-016 and SCI P354 both recommend response factor (RF) criteria which is a multiple of the base curve shown in Figure 10 representing a minimum vibration magnitude for approximately equal human response with respect to human annoyance to continuous vibrations (BSI 2008a). The RF defines an ‘acceptable level’ of vibration for various occupancies. BS 6472 states that adverse comments of vibration are rare for vibration magnitudes below the base curves, however, this does not imply that ‘annoyance and/or complaints are necessarily to be expected at higher magnitudes (BSI 2008a). This highlights the subjectivity of human perception of floor vibrations and the importance of selecting criteria based on the expected occupation and occupant activity.

The RF based on weighted RMS acceleration is typically calculated as per Equation 3.21. The 0.005 m/s^2 in the denominator refers to the baseline perception threshold for the most sensitive frequency range of 4 to 8 Hz. Although SCI-P354 uses Equation 3.21 for evaluation of both resonant and transient responses, CCIP-016 is the only guide which acknowledges that after 8 Hz, human perception threshold is based on constant velocity.

$$RF = \frac{a_{w,rms}}{0.005} \tag{3.21}$$

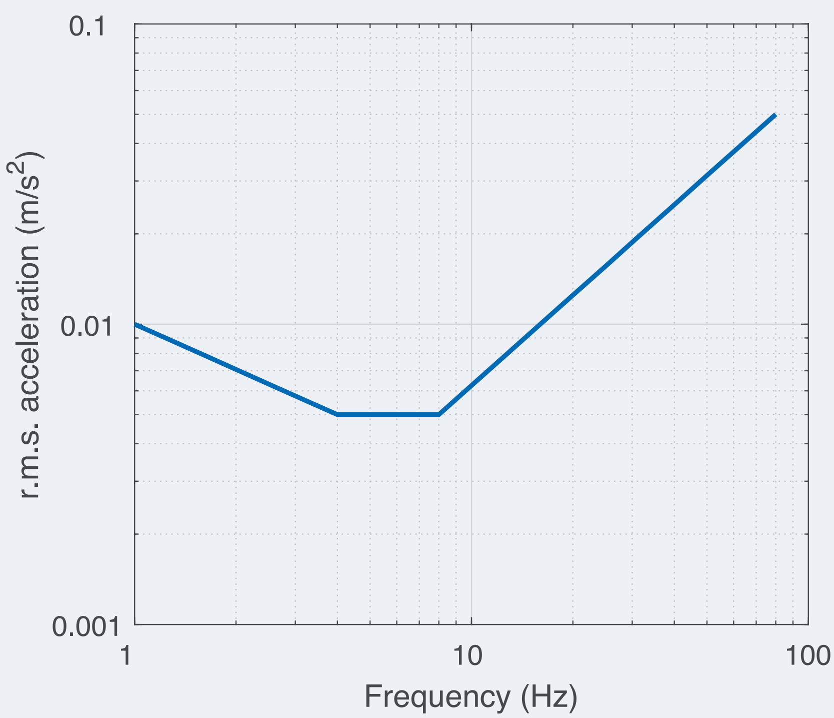


Figure 10: Building vibration z-axis base curve for acceleration (foot-to-head vibration direction) (British Standards Institution 2008).

Vibration Dose Value

A cumulative measure, such as the Vibration Dose Value (VDV), has been suggested to be more appropriate in assessing vibrations from human walking (Ellis 2001). The VDV places importance on the amplitude of vibration and effectively relaxes the response limit of those specified for continuous vibrations, but only for short periods of time when the large amplitudes occur. SCI P354 suggests that if the floor does not satisfy the conservative RF criterion, a representative VDV value as per research from Ellis (2001) can be checked:

$$rVDV = 0.68a_{w,rms}\sqrt[4]{n_a T_a} \quad (3.22)$$

where n_a = number of times the activity will take place in an exposure period; T_a = duration of an activity i.e. time taken to walk along a corridor (s). Ellis (2001) suggests that three possible scenarios thzt may be considered in calculating $n_a T_a$:

- Extremely busy scenario: a person crossing the floor every second for a 16-hour day.
- Busy scenario: as a person walking across the floor every minute for an 8-hour day.
- Quiet scenario: one person walking across the floor 4 times per hour for an 8-hour day.

Otherwise, Equation 3.22 can be rearranged based on the VDV limits provided in BS 6472-1 (2008) to identify the maximum number of times that activity will can occur in an exposure period.

3.5.4 Checking Response against Acceptance Criteria

Response factor limits

Table 6 summarises the performance criteria for various floor occupancies as suggested by CCIP-016 and SCI P354 with comparison to those suggested by BS 6472 (BSI 2008a). Human perception of vibration is not only influenced by amplitude but also by frequency and duration of the walking (Willford, Young & Field 2007). The criterion is based on a single person walking at the most critical footfall rate. Compared to CCIP-016 and BS 6472, SCI P354 has the most relaxed limit for commercial floors with a RF of 8. CCIP-016 makes note of different types of commercial spaces and has reduced the RF by a factor of 2 for many typical office scenarios. These values are in line with those suggested by BS 6472. Another difference of CCIP-016 limits is the consideration of partitions where the RF can be relaxed by a factor of 1.5 for areas which have many full-height partitions that have not been previously considered in the prediction analysis. Although DG 11 provides a response limit of 0.5% gravity for offices and residences, an equivalent RF limit is 7.

As the criteria is based on human perception of vibration magnitudes, the limiting value indicates a level of vibration at which probability of adverse comment is low (but not zero probability) (Willford & Young 2006). If these limits were doubled, adverse comment may result. This is where vibration limits can lead to some uncertainty in determining satisfactory behaviour and it is the responsibility of the engineer and client to balance risk and cost to agree on a reasonable limitation. Note that a floor having RF of 3.8 would be perceived similarly to one with RF of 4.1, i.e. these limits should not be used as a pass/fail but rather as an indication.

Table 6: RF criterion for various floor occupancies from CCIP-016, SCI P354 and BS 6472.

Environment		**Response Factor**		
		SCI P354	**CCIP-016**	**BS 6472**
Critical working areas		1	1	1
Residential	Day	-	4 – 8	2 – 4
	Night	-	2.8	1.4
Commercial	Premium quality open-plan offices and when precision tasks are to be undertaken[1]	8	4	4
	Open-plan offices with busy corridor zones near mid-span[1]	8	4	4
	Heavily trafficked public areas with seating[1]	8	4	4
	Other commercial buildings not covered by the above categories*	8	8	4
Retail	Shopping mall	4	-	-
	Dealing floor	4	-	-
Stairs	Light use (e.g. offices)	32	-	-
	Heavy use (e.g. public buildings, stadia)	24	-	-

Notes:

[1] Target can be relaxed by a factor of up to 1.5 if there are many full-height partitions that were not explicitly included in the prediction analysis.

* intermittent vibrations

\- not provided

Vibration Dose Value limits

VDV limits are more holistic in that the response is classified based on probability of adverse comment. These limits reproduced from BS 6472-1 (2008) in Table 8 can be adjusted with the multiplying factors in the final column of Table 7 for various occupancies.

Table 7: RF criterion for various floor occupancies from CCIP-016, SCI P354 and BS 6472.

Place	**Low probability of adverse comment**	**Adverse comment possible**	**Adverse comment probably**
Residential buildings 16 h day	**0.2–0.4**	**0.4–0.8**	**0.8–1.6**
Residential buildings 8 h night	**0.13**	**0.26**	**0.51**

Hu and Chui's limit

Hu and Chui (2004) undertook a field test program involving 130 timber floors in order to develop an improved design method using 'designer-useable formulas' to control vibrations for wood-based floor constructions. Occupants' perception to vibrations were correlated to the measured parameters including static deflection under 1kN, natural frequency, initial velocity and acceleration and RMS acceleration. From the correlation, it was decided that a 1 kN static deflection (*d*) and natural frequency (*f*) were the most suitable design parameters, simply due to the ease of use from the designer and ease of measurement with acceptable accuracy. Through regression analysis, the following formula was proposed:

$$\frac{f}{d^{0.44}} > 18.7 \tag{3.23}$$

The formulas for deflection under a point load at mid-span and frequency were derived from the ribbed-plate theory and considered semi-rigid connections between web and flange, torsional rigidity of joists and sheathing stiffness in the span and across-joist directions (Hu & Chui 2004). Construction details shown to enhance performance, such as between-joist bridging, strong-back and strapping, are also accounted for in the formulas. More information can be found in Hu & Chui.

The method was validated through data obtained for 106 timber floors built with wood I-joists and solid sawn timber joists with spans of 3–13 m. Depths of floor joists ranged from 140 to 450 mm. The subjective ratings of these floors were compared to the proposed acceptance criteria, (see Figure 11). It was observed that the method is quite effective in differentiating between unacceptable and acceptable floors and shows 'great potential to properly address issues which are deemed to be problematic with the design approaches' including long-span floors (Hu & Chui 2004).

Main differences to note from Hu and Chui's assumptions compared to response of ribbed deck cassette floors are, firstly, the criterion only accounts for the first fundamental vibration mode. This neglects the fact that the second mode which, as mentioned earlier, may be close to the first mode and may have more contribution to the floor response. Secondly, ribbed deck cassettes act in a composite manner rather than the semi-composite behaviour assumed. Although more investigation needs to be undertaken as to how this criterion applies to ribbed deck cassette construction, it should be kept in mind as a possible limitation measure.

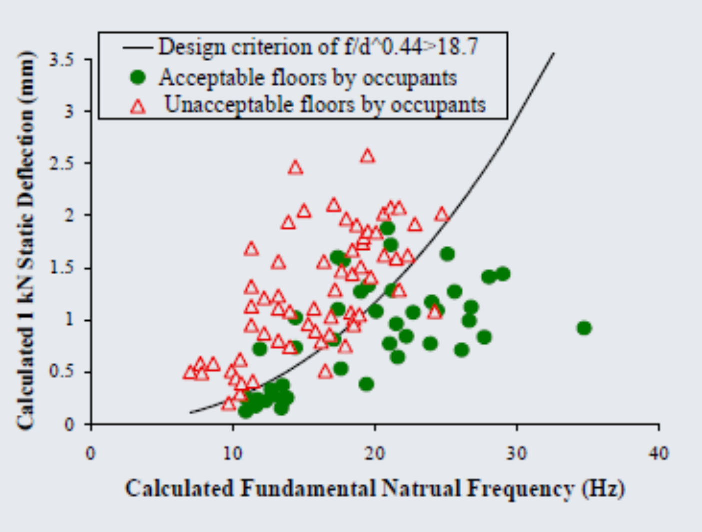


Figure 11: Comparison between subjective evaluation and proposed criterion (Hu & Chui 2004).

3.6 Modelling the floor using finite element

Finite element modelling is a useful tool to accurately analyse the modal properties of more complex or irregular structures. Typically, a complete floor plan is modelled including columns and shear walls, which represents a more realistic situation than a simplified analysis. A transient analysis can also be undertaken to identify locations of high response due to certain nodes being excited. The following modelling details should be considered for dynamic assessment of a ribbed deck cassette floor using FEM.

- **Element type:** Shell elements based are recommended to be used for the flange. The web can be modelled as an isotropic beam or shell element, depending on the desired complexity of boundary connection. Note that a Timoshenko beam element or Mindlin shell element should be used to take into account the shear deformation. Using shell elements for the web means that there are a number of nodes through the depth as opposed to one node if it was a beam element. This provides more flexibility when connecting back into the main structure. If modelling as a beam element, the flange element should be offset from the flange centreline.
- **Flange to web connection:** Web elements should be rigidly tied into the flange elements, i.e. nodes can be coincident between web and flange. Under service loads, the section can be considered to act as a fully composite section.
- **End boundary conditions**: Support conditions where the overhanging flange is secured to the primary beams with screws can be assumed to act as pins. The position of screws (i.e. edge distance) should be followed in the model to ensure the effective length is accurate. Further investigation is required for other support conditions.
- **Cassette-to-cassette connections**: Adjacent cassettes are connected through both web and flange members. Screws connecting web members can be assumed to act as translationally coupled nodes. For flange-to-flange connections such as a splice or diagonal screws, nodes at screw locations can be assumed to provide translational restraint but should be rotationally free. Further investigation is required to confirm whether rotational restraint can be considered in a numerical model.
- **Additional non-structural mass:** Parametric studies will be undertaken to determine whether strategic positioning of mass will reduce floor response.
- **Material property**: All material properties should be input from the manufacturer's technical data sheet.

- **Continuity over primary beam:** From a review of literature on continuous composite floor systems, such as steel-concrete (Ellis et al. 2010; Pavic et al. 2007; Zheng et al. 2010) and timber-concrete (Ghafar et al. 2010), it is expected that there will be little difference to modal properties when compared to a simply supported cassette. When concrete is poured continuously over the primary beams, small cracks occur in the tensile zone above the primary beams. This substantially decreases the stiffness of the section compared to analytical models of a 'continuous' floor. If small cracks in concrete can result in a not-perfect continuous system, then it is more than likely that the panel-to-panel connection between ribbed deck cassettes will create a similar situation.
- **Modelling of elastomer material:** Elastomers at support locations may be modelled as a spring-damper system.
- **Other structural considerations:** Column, shear walls, and façade restraints should be included in the model to ensure additional stiffness is accounted for. Voids in the floor plan must be modelled as floors around openings are often more susceptible to floor vibration.

3.7 Construction Considerations

Services and insulation can be incorporated into the floor structure by utilising the space or void between joists. For cassettes with a top flange, the floor is immediately accessible for fit-out works and suspended ceiling installation. For bottom flange cassettes, workers have direct access to the voids which can be used for a raised floor or other required services. However, care should be taken as the joists pose a trip hazard to workers. Adjacent panels can be connected through diagonal screws (Figure 12a) or splice plate (Figure 12b). The splice plate will typically be recessed into the panel so as to not affect finishes and is less time consuming on site.

Testing on two adjacent ribbed deck cassettes with varying connection systems with screw spacing of 300 mm and 150 mm led to the following observations:

- **Web-to-web connection:** Although the natural frequency of the first bending mode remained the same from a single cassette, the torsion mode frequency reduced by about 3 Hz to become the first mode. Damping ratio increased for the first torsion mode by about 1%. Some reduction in Response Factor from 300 mm to 150 mm spacing.
- **Splice connection + web-to-web connection @ 150 mm c/c:** Negligible change in Response Factor from 300 mm to 150 mm spacing.
- **Diagonal screw connection + web-to-web connection @ 150 mm c/c:** Negligible change in Response Factor from 300 mm to 150 mm spacing. Negligible difference in response between splice and diagonal screw connections.

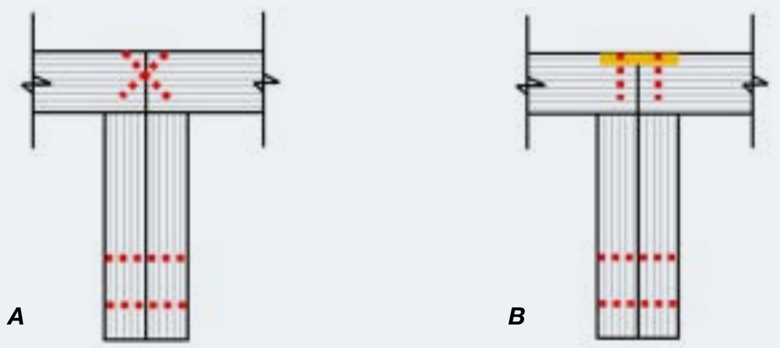


Figure 12: Panel-to-panel connection options for ribbed deck floor.

3.8 Discussion on Response Factor Results for Ribbed Deck Floor

Appendix A Section A.1.7 shows that CCIP-016, SCI P354 and DG 11 procedures significantly overestimated the response of the floor. CCIP-016 had the smallest margin of error for a 2.0 Hz walking pace at 221%; for 2.14 Hz walking pace, DG 11 produced the smallest margin of error at 115%. SCI P354 had the largest margin of error in both cases at 282% and 158% for 2.0 Hz and 2.14 Hz pace frequency, respectively. These errors are from a case where the measured modal properties have been used. This highlights that the predicted effective impulse equation determined by Willford et al. (2006) from experiments of single footfall time histories by Kerr (1998) may not accurately represent the forces occurring on a timber floor. Reasons for the inaccuracy may be a result of:

- **Human-structure interaction:** Some research has shown that the ground reaction forces from a single pedestrian on a more flexible floor surface, such as a footbridge, are less than on a rigid surface (Caprani et al. 2015; Zivanovic, Pavic & Reynolds 2005).

- **Inter-subject variability**: The predicted footfall loading equations do not consider intra-subject variability (differences in response of an individual from one moment to the next). Studies have shown that the slight differences in stride length and frequency between left and right legs have shown to generate subharmonics where excitation energy leaks between the bands of main harmonics (Sahnaci & Kasperski 2005) and becomes more pronounced for higher harmonics (Brownjohn, Pavic & Omenzetter 2004).
- **Time dependency of Response Factor:** There is some conservatism in assuming that the maximum RF will occur for a continuous period. In reality, the RF may only occur for a few seconds when a subject walks across the floor. Through a proposed stochastic model of the footfall impulse, a statistical approach of the RF methodology was investigated (Živanovié & Pavié 2009) indicating a probability of exceedance of a certain RF. Response measures in Japan such as the VLT is another example of consideration of time duration of response above a certain threshold (for measured data only) (Matsushita et al. 2015).

3.9 Recommendations

The following points should be considered when designing ribbed deck cassette floors.

- Shear deformation should be considered in design.
- Timber cassette floors may be susceptible to closely spaced modes which may interact to produce higher responses. The second mode may have a higher modal participation depending on the location of excitation. Floor assessment procedures considering only the fundamental mode may not be appropriate for long-span timber cassette floor systems.
- A damping ratio of 1% should be used until further investigations are undertaken on ribbed deck floor systems.
- The human loading model proposed in design guides (CCIP-016, SCI P354 and AISC DG 11) do not consider important aspects such as human-structure interaction, inter-subject variability and the time varying nature of the RF.
- Negligible difference in modal properties and no clear trend of floor response between the two flange-to-flange connection types (splice and diagonal screws) as well as for reduction of screw spacing from 300 mm to 150 mm.
- A finite element model is recommended to determine modal properties in order to capture torsional modes.
- Support conditions to primary beam should be modelled as a pin-support until further investigation confirms otherwise.

4 Plate type floor using cross-laminated timbers

4.1 Overview

Cross-laminated timber (CLT) is solid timber engineered wood product (EWP) capable of spanning whole walls and floors. The component elements of CLT are boards of timber that are laid side by side to form a plate layer. Each layer is laid 90 degrees to the adjacent layer forming a solid wood panel that improves the structural properties of the timber. Figure 13 shows the general arrangement of a CLT plate.

This document presents the current design procedures available for CLT. It is written in conjunction with experimental work on the vibration characteristics of CLT being undertaken at UTS (final project report, PNA 341-1415, available on the FWPA website).

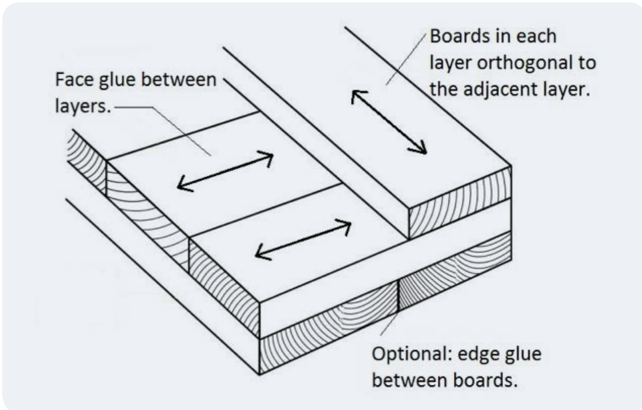


Figure 13: CLT panel arrangement: stacked panels of 5-layered CLT (left). A schematic of a 3-layered panel (right).

4.2 Design Considerations and Scope

Timber is a highly workable material with many options and possible configurations. Therefore, this design guideline is limited to the following design parameters that are within the practical limits for spanning a CLT floor up to 9 m span.

- **Spans up to 9 m**. While CLT is structurally efficient for spans up to around 6 m it can span further. Steel and concrete are capable of structurally satisfying a building with a 9 x 9 m column grid. This column grid is desirable in commercial buildings for efficient desk spacing in the office space and car parks in the basement level. This design guide includes strategies to allow CLT to satisfy a 9 x 9 m column grid.
- **Single span**. Manufacturing and transport capabilities for CLT limit the panel length to 12 m. Therefore, double spans of 6 m are economically efficient. However, there are cases where longer spans are required. This document suggests methods and design guidelines for a single non-continuous span of 9 m.
- **Framing**. While timber framing is desirable, both steel and concrete framing could provide the structural support. Framing options for providing one-way and two-way support are considered. Layouts for these framing options are shown in Figure 14.

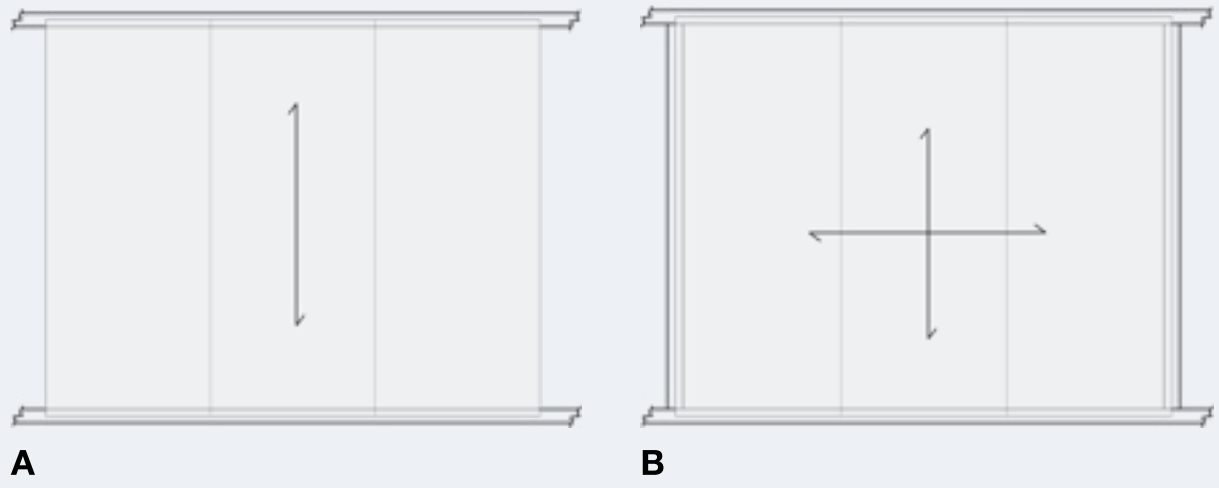


Figure 14: Framing options for CLT floors (a) CLT panels spanning one-way between primary beams and (b) CLT panels spanning two-way between primary beams and secondary beams.

- **Panel-to-panel connections.** Panel-to-panel connections are necessary to facilitate a 9 x 9 m grid. For a 9 m column grid there will be three or possibly four panels making up the cross-section of the floor plate. A half-lap connection is one of the most common connections (Figure 15a). Another common connection type is a single surface spline. This connection involves an added length of timber that can be recessed into the panel (shown not recessed) and screwed to each panel (Figure 15b).

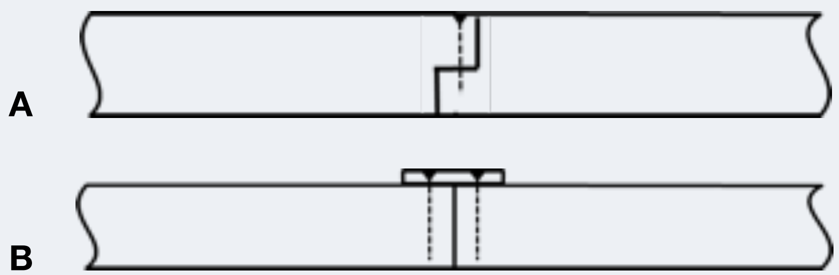


Figure 15: Panel-to-panel connection types: (a) a half-lap connection (top) and (b) a single surface spline (bottom).

- **Floor-to-supporting element connection.** Simple screws connecting the CLT floor to the supporting beam or wall in a single row were investigated. A self-tapping partially threaded screw is drilled into the timber from above at a set spacing. An end-span connection is illustrated in Figure 16. This connection can be extended to span multiple floors by supporting the floors on a wide support beam or wall (Figure 17a) and adding bending moment continuity to the floor by an additional top plate (Figure 17b).

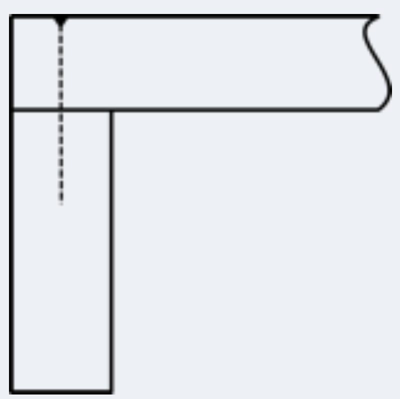


Figure 16: Floor-to-wall support connection with a single partially threaded self-tapping screw.

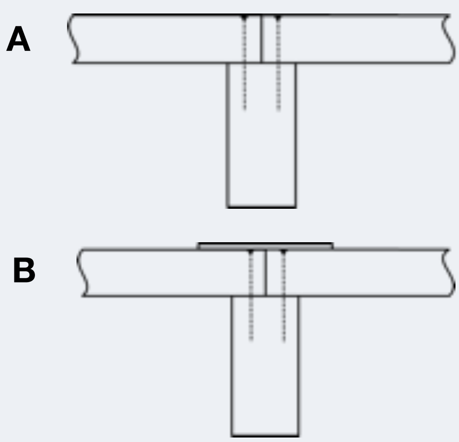


Figure 17: Floor-to-wall support connection for multiple span floors (a) without top plate and (b) with top plate.

4.3 Design Requirements

The design requirements for CLT floors can be divided into two stages: evaluation of the strength capacity and assessment of the serviceability limit. The design criteria can be summarised as:

Strength Design:

- bending, shear and bearing strength for vertical loads
- design for in-plane strength if diaphragm action present
- fire and earthquake design.

Serviceability Design:

- short-term deflection
- long-term deflection
- vibration.

Due to the high strength-to-weight ratio of timber, serviceability generally governs the design of CLT floors. For this reason most of the research to-date has focused on the serviceability design of CLT (Gagnon & Pirvu, 2011). This document presents current research and methods on the strength and serviceability design for CLT floors.

4.4 Design Procedure

4.4.1 Material Property Considerations

Characteristic strength values for engineered wood products (EWPs) such as glued laminated timber (Glulam) and laminated veneer lumber (LVL) are determined from experimental testing. However, the problem with this approach for CLT is that there are numerous possible layups, material types and configurations. A standard released by the American National Standard ((ANSI 2012), has categorised CLT into grades and provided the respective strength values for a limited number of cross-section sizes. This approach has also been adopted by some manufacturers for the products they regularly produce. The design methods presented in this document, however, use the characteristic values of the lumber that makes up a CLT panel. Therefore, design calculation of any configuration of CLT, material type, thickness, and number of layers and be used. Characteristic values and test configurations for CLT are presented by Unterwieser and Schickhofer (2014).

An important characteristic of CLT is that it cannot be viewed as a homogenous material due to a phenomenon known as rolling shear. This occurs in CLT due to the low shear capacity in the radial and tangential directions of timber and is an important consideration for CLT design of (Figure 18). Rolling shear can contribute significantly to a panel's deflection under bending due to the shear deformation of the transverse layers.

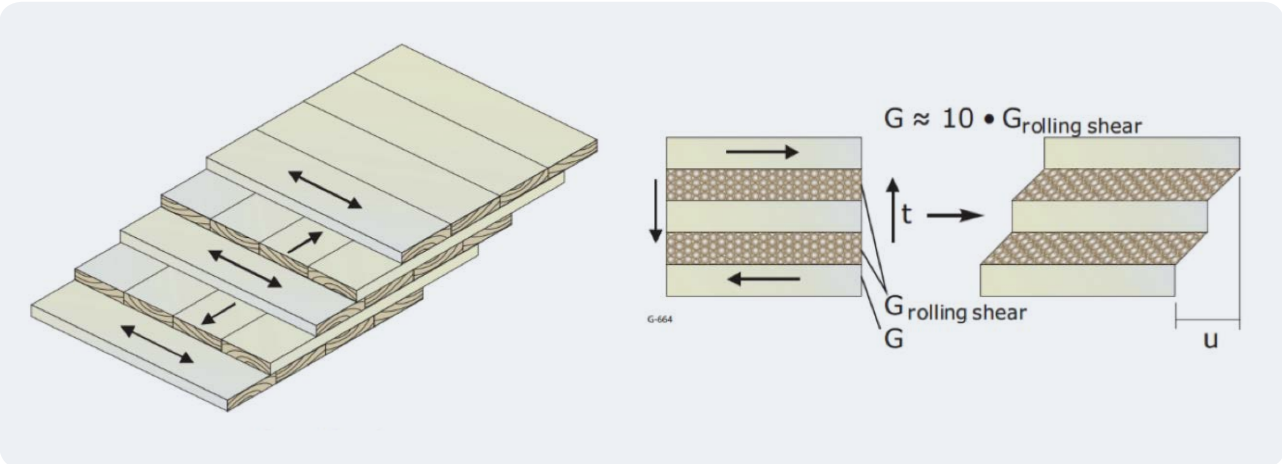


Figure 18: The effect of the low shear strength of the transverse layers in a CLT panel (Gagnon & Pirvu, 2011).

While more research is needed to provide values for rolling shear modulus of various timber species, experimentation to date indicates the shear modulus (G0) to be between 1/12 and 1/20 of the true modulus of elasticity and the rolling shear modulus (GR) to be 1/10 of the shear modulus (Gagnon & Pirvu 2011).

4.4.2 Current Design Guidelines

Several design guidelines have been published that present the holistic design of CLT. European research groups have been the leaders in design and manufacturing of CLT. Design software for CLT has been released by the University of Graz, called CLTDesigner, the calculation methods for this software are presented in this document. More recently the German-Czech company Dlubal has integrated a CLT module into its RFEM software that provides the structural analysis for CLT.

Another important document is the CLT Handbook released by FPInnovations in Canada (Karacabeyli & Douglas 2013). It provides comprehensive documentation of the manufacturing, design and construction of CLT.

Four methods for calculating the strength and serviceability properties for timber are considered in this document; CLTdesigner, Gamma method, composite k method and the shear analogy method. These methods calculate the design capacity for timber structures using a modified version of the Euler-Bernoulli hypothesis of plane sections remain plane. For CLT floors where the ratios of the length/thickness ≥ 15 all these methods converge (Thiel, 2014). Therefore, in such situations, the designer has the option to choose among these methods to calculate the properties of the CLT section depending upon relevant code and design requirements.

These theories are limited as the analytical models are based on beam theory, whereas CLT is a plate element. A more advanced examination is recommended in cases of large point loads, for accounting two-way spanning effects and for length/thickness ratio less than 15. Advanced laminated plate theories requiring higher computational input have been developed for such cases (Thiel, 2014).

Table 8 summarises these methods as well as the material and capacity factors from AS 1720.1 (2010).

Table 8: Summary of available methods for determining the design of CLT.

Method	Design Process	Properties calculated
AS 1720.1 (Standard 2010)	Factors for bending Factors for shear Factors for bearing	AS 1720.1 is used to determine the capacity and modification factors and the characteristic strengths.
CLTDesigner (Thiel 2013)	Bending strength Shear strength Bearing strength Bending stiffness Shear stiffness Vibration	Section modulus, Z Effective area, A_{eff} Effective stiffness K_{CLT} Shear stiffness S_{CLT} Frequency, acceleration
Gamma Method (Eurocode 2003)	Bending strength Shear strength Bending stiffness	Section modulus, Z Effective area, A_{eff} Effective stiffness, EI_{eff}
Composite K Method (Gagnon & Popovski 2011)	Bending strength Bending stiffness	Section modulus, Z Composite factor, k_1
Shear Analogy Method (Gagnon & Popovski 2011)	Bending strength Bending stiffness Shear stiffness	Section modulus, Z Effective bending stiffness, EI_{eff} Effective shear stiffness, GA_{eff}

4.4.3 Strength

The timber for the FWPA-funded project PNA 341-1415 was sourced from New Zealand, so the material and safety factors in AS 1720.1 (2010) are applicable for calculating the strength design capacity. However, AS 1720.1 (2010) does not present methods to determine the relevant moment of inertia, section modulus and effective area calculations for CLT. Due to the cross lamination of CLT, the reduction in these values due to the shear slip is accounted for by a number of methods presented in this section. This section is organised to first present the relevant 'k' values from AS 1720.1 and then presents the methods from international guidelines and research to calculate the section properties.

Both the bending and shear strength are required to be assessed under ultimate limit state loads for strength, earthquake, and fire. Additionally, it is important to check the bearing strength of CLT. This is due to the low compression strength of timber when loads are applied perpendicular to the grain. It is particularly important for the design of CLT buildings where the floors extend between the walls (Figure 19).

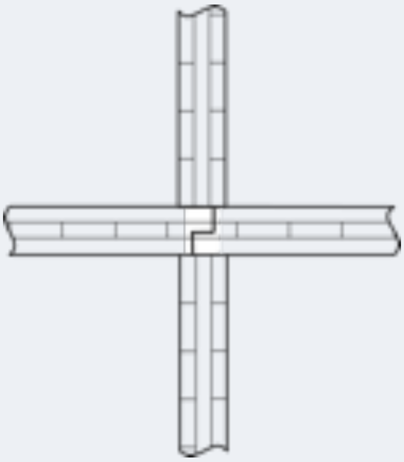


Figure 19: CLT building construction; floors sandwiched between wall plates.

Factors for bending strength AS 1720.1

The Australian standards calculate the bending moment capacity for timber structures using the Euler-Bernoulli hypothesis of plane sections remain plane. Where Z is the section modulus of the timber cross-section with for a rectangular joist can be simply calculated as the moment of inertia divided by the centroid. The section modulus for CLT, Z_{CLT} is calculated using one of the methods presented in this document as AS 1720.1 does not provide guidance for CLT.

$$M_d = \varphi k_1 k_4 k_6 k_9 k_{12} f'_b Z_{CLT} \qquad (4.1)$$

φ	safety factor equal to 0.95 for secondary members in structures other than houses
k_1	accounts for load effects, equal to 0.57 for permanent loading
k_4	accounts for moisture content, generally equal to 1.0 unless there is significant moist environment or where partial seasoning occurs.
k_6	accounts for temperature effects, 1.0 for covered timber under ambient conditions
k_9	strength sharing factor for Glulam is taken as unity; could be as high as 1.33 for CLT
k_{12}	stability factor to be taken as unity for CLT due to the low thickness-to-width ratio
f'_b	the characteristic bending strength of timber, for CLT, $f'_b = f_{m,CLT,k}$, see section 4.4.3 CLTDesigner.

Factors for shear strength AS 1720.1

The shear strength of a beam is generally more complicated to calculate, as unlike the bending stress distribution, the shear stress is not linear. For a rectangular and homogenous cross-section of a beam the shear strength is simple to calculate as the shear area is equal to 2/3 the gross area. Due to the non-homogenous cross-section of CLT the method provided in section 4.4.3 is recommended to calculate the shear plane area.

$$V_d = \varphi k_1 k_4 k_6 f'_s A_s \qquad (4.2)$$

A_s	is the shear plane area which is for a non-composite rectangular section 2/3 of the gross area. For CLT $A_s = A_{eff}$ discussed in section on CLT.
f'_s	is the characteristic shear strength, for CLT the shear strength at mid-section, $f_{v,CLT,d}$ and rolling shear strength $f_{r,CLT,d}$ of the timber are 3.0 N/mm² and 0.7 N/mm² respectively (Unterwieser & Schickhofer, 2014). If edge bonding has occurred in the manufacturing the rolling shear strength can be increased to 1.25 N/mm².

The k modification factors are the same as those for bending strength.

Factors for bearing strength AS 1720.1

To calculate the bearing strength the area of applied load, Ap, and the characteristic bearing strength of the timber, f'_p is required. The strength of bearing calculated as:

$$N_{d,p} = \varphi k_1 k_4 k_6 k_7 f'_p A_p \qquad (4.3)$$

k_7 accounts for the location of the bearing position. Location at edge of timber piece is given as unity. A location factor specific for CLT, $k_{c,90,CLT}$ is included in the CLTdesigner section below.

f'_p For CLT is given as 2.85 N/mm² (Thiel, 2014).

The other modification factors are the same as those for bending strength.

CLTdesigner

Bending strength

The software program CLTdesigner developed by the Centre of Competence (holz.bau.forschungs.gmbh) in Graz, Austria, uses the Bernoulli-hypothesis of plane sections remaining plane to calculate the bending strength. The program assumes there is negligible bending stress in the cross layers. This is due to the cross layers being orientated so that the weak grain of the timber contributes to the cross-section stiffness. Further, the transfer of normal stresses in the cross layers is likely not possible due to lack of edge gluing (Thiel 2013). The bending stress distribution for the longitudinal and transverse layers is shown in Figure 20, and the bending stiffness, K_{CLT} is calculated using Equation 4.4.

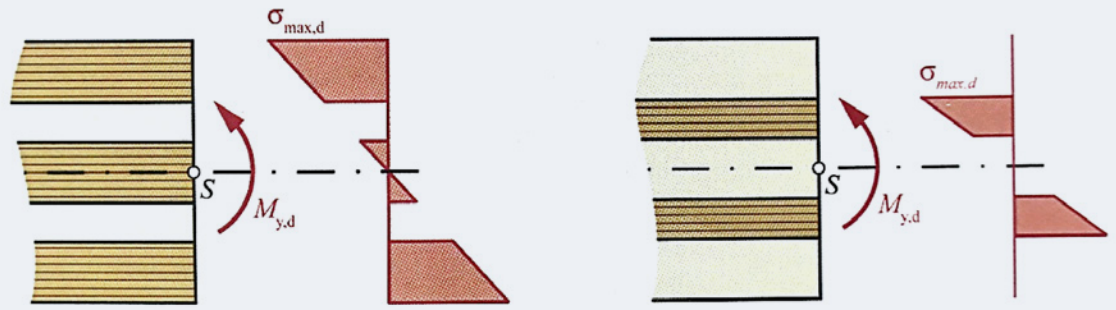


Figure 20: The normal stress distribution of a CLT panel for the bending moment of a floor panel for longitudinal bending (left) and transverse bending (right) (Thiel 2013).

$$K_{CLT} = \sum (E_i I_i) + \sum E_i A_i z_i^2 \qquad (4.4)$$

E_i the elastic modulus of the *i*th layer

I_i the moment of inertia of the *i*th layer

A_i the area of the *i*th layer

z_i the distance from the centroid of the *i*th layer to the centroid of the entire cross-section.

The maximum stress, $\sigma_{max,d}$, of the cross-section is calculated using Equation 4.5.

$$\sigma_{max,d} = \frac{M_{y,d}}{K_{CLT}} \frac{t_{tot}}{2} E_1 \qquad (4.5)$$

$_{tot}$ the total thickness of the CLT panel

E_1 the elastic modulus of the outermost layer

To calculate the bending design stress of CLT, $f_{m,CLT,d}$, two methods are suggested, this first is based on the tensile strength of the timber and the second on the Glulam product with an equivalent strength grade. The tensile strength value is presented here as it's more easily translated to the base timber material properties. The characteristic values that are used for CLT are presented in more detail by Unterwieser and Schickhofer (2014).

$$f_{m,CLT,k} = k_{m,CLT} f_{t,0,1,k}^{0.8} \tag{4.6}$$

$k_{m,CLT}$ is equal to 3 for timber with a tensile strength CV of 25% and 3.5 for a CV of 35%

$f_{t,0,1,k}$ is the characteristic tensile strength of the timber

Rearranging Equations (4.5) and (4.6), we find an expression for the design bending moment My,d:

$$M_{y,d} = \frac{2K_{CLT}}{t_{tot}E_1} f_{m,CLT,k} \tag{4.7}$$

Equation 4.7 is in the form of Euler Bernoulli's beam theory, $M_{y,d} = fZ_{CLT}$, where Z_{CLT} is the section modulus for cross-laminated timber calculated using Equation 4.8. This section modulus and the value for design bending stress $f_{m,CLT,k}$ can be used to calculate the bending moment capacity in accordance with AS 1720.1.

$$Z_{CLT} = \frac{2K_{CLT}}{t_{tot}E_1} \tag{4.8}$$

Shear Strength

CLTdesigner uses the classical procedure for unidirectional layered cross-sections to calculate the shear stress distribution given by Equation 4.9. The assumption that $E_{90} = 0$ means that there is no shear stress increase in the cross layers (Figure 21).

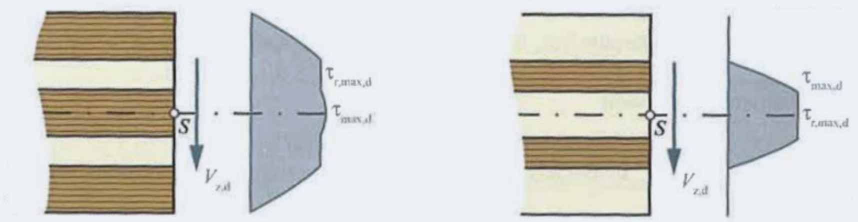


Figure 21: The shear stress distribution in a CLT cross-section for shear caused by longitudinal bending (left) and transverse bending (right), (Thiel 2013).

$$\tau = \frac{V_z \int_A E(z).z.dA}{K_{CLT}.b(z_0)} \tag{4.9}$$

The shear stresses need to be assessed for both the rolling shear stress, $\tau_{r,max,d}$ (at the inter layers) and the maximum shear stress $\tau_{max,d}$ at the centre of the CLT panel and therefore satisfy Equation 4.10.

$$\frac{\tau_{max,d}}{f_{v,CLT,d}} \leq 1.0 \quad \text{and} \quad \frac{\tau_{r,max,d}}{f_{r,CLT,d}} \leq 1.0 \tag{4.10}$$

Solving Equation 4.9 for a 5-layered CLT plate and combining with Equation 4.10 an expression for shear force at the mid cross-section (4.11) and at the rolling shear layers (4.12) is given. These equations need to be multiplied by appropriate material and safety factors.

$$V_{mid} = f_{v,CLT,d} \frac{K_{CLT}}{\left(E_1 t_1 z_1 + \frac{E_3 t_3^2}{8}\right)} \tag{4.11}$$

$$V_{rolling} = f_{r,CLT,d} \frac{K_{CLT}}{(E_1 t_1 z_1)} \tag{4.12}$$

Unless experimental testing has occurred, the values for the shear strength $f_{v,CLT,d}$ and rolling shear strength $f_{r,CLT,d}$ of the timber are currently 3.0 N/mm^2 and 0.7 N/mm^2 respectively (Unterwieser & Schickhofer, 2014). If edge bonding has occurred in the manufacturing, the rolling shear strength can be increased to 1.25 N/mm^2.

Rearranging Equations 4.11 and 4.12, the effective area for shear strength at the mid-point of the section and shear at the transfer layers are given by:

$$A_{eff,mid} = \frac{K_{CLT}}{\left(E_1 t_1 z_1 + \frac{E_3 t_3^2}{8}\right)} \tag{4.13}$$

$$A_{eff,rolling} = \frac{K_{CLT}}{(E_1 t_1 z_1)} \tag{4.14}$$

K_{CLT} is the bending stiffness in Equation 4.4

$E_{1,t1}$ are the elastic modulus and thickness of the outer layer of a 5-layer CLT panel

z_1 is the distance between the centroid of layer 1 and the centroid of the entire cross-section

$E_{3,t3}$ are the elastic modulus and thickness of the middle layer of a 5 layer CLT panel

Bearing Strength

The bearing strength is calculated by multiplying the contact area with the characteristic compressive strength of CLT perpendicular to the plane of the CLT floor. CLTdesigner uses a characteristic value of $f_{c,90,CLT,k}$ = 2.85 N/mm^2 which has been determined from testing. This value must be multiplied by the appropriate modification factors (see Factors for bearing strength AS 1720.1). Table 9 gives the appropriate multiplying factors to account for the location of bearing, k_7.

Table 9: Factor to account for location of bearing (Thiel, 2014).

Load Type	Load Location	k_7
Point	Central (away from edge)	1.8
Point	Edge of panel (not a corner)	1.5
Point	Corner	1.3
Line	Central and parallel to span	1.3
Line	Central and perpendicular to span	1.8
Line	Edge and parallel to span	1.0
Line	Edge and perpendicular to span	1.5

Gamma method

Bending strength

The gamma method has been developed from mechanically jointed beam theory and is detailed in Eurocode 5 and in the CLT Handbook by FPInnovations. Therefore, only limited equations are presented in this document.

To calculate the effective bending stiffness using the gamma method the reduction in stiffness due to shear slip is accounted for by a stiffness component (γ_i) which is calculated using Equation 4.16. The effective stiffness can then be determined using Equation 4.15.

$$EI_{eff} = \sum_{i=1}^{n} \left(E_i I_i + \gamma_i E_i A_i z_i^2\right) \tag{4.15}$$

$$\gamma_i = \frac{1}{1 + \pi^2 \frac{E_i A_i}{l^2} \cdot \frac{t_1'}{G_R b}} \tag{4.16}$$

G_R, is the rolling shear modulus

b is the width of the cross-section

A is the area of layer i

E is the elastic modulus of layer i

l is the length of the floor

t'_1 is the thickness of the slip layer

The section modulus of bending moment is calculated from Equation 4.17 and can be used to determine the bending moment capacity of CLT.

$$Z_\gamma = Z_{CLT} = \frac{(EI)_{eff}}{E_1(\gamma_1 z_1 + 0.5t_1)} \quad (4.17)$$

z_1 is the distance from the centroid of the cross-section to the centroid of the outer layer.

Shear Strength

Shear stresses are calculated using mechanically jointed beam theory. The difference between the CLTDesigner method and the Gamma method is that the latter includes the shear strength contribution of the cross layers. The resulting effective shear area for the mid-point of the cross-section and at the transverse layers are given by:

$$A_{eff,mid} = \frac{\left(EI_{eff}\right)b}{\left(\gamma_1 E_1 A_1 z_1 + E_1' A_1' z_1' + \gamma_2 E_2 \frac{A_2}{2}\frac{t_2}{4}\right)} \quad (4.18)$$

$$A_{eff,rolling} = \frac{\left(EI_{eff}\right)b}{\left(\gamma_1 E_1 A_1 \left(z_1 - \frac{t_2}{2}\right) + E_1' A_1' \left(z_1' - \frac{t_2}{2}\right)\right)} \quad (4.19)$$

Composite method

Composite theory was developed for calculating the bending strength of plywood. From composite theory, the design bending moment is calculated using Equation 4.20. The composite factor, k, is a value that accounts for the reduced stiffness of the entire cross-section due to the transverse layer's flexibility. For a CLT floor with the outer layers running longitudinal to the span the value for k is given by Equation 4.21. Any shear deformation is not considered using the k-method.

$$M_{y,d} = \varphi k_1 f_{b,eff} S_{gross} \quad (4.20)$$

$$k_1 = 1 - \left[\left(1 - \frac{E_{90}}{E_0}\right)\left(\frac{a_{m-2}^3 - a_{m-4}^3 + \cdots \pm a_1^3}{a_m^3}\right)\right] \quad (4.21)$$

The value for a_m is shown in Figure 22. E0 is the elastic modulus of the longitudinal layers and E90 is the elastic modulus of the transverse layers. The elastic modulus relationship is given as E90 = E0/30.

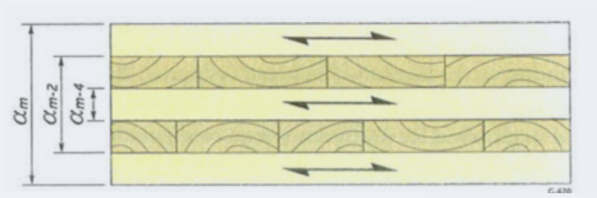


Figure 22: Cross section values for calculation of value k using composite theory.

The section modulus for bending strength for composite theory is given by:

$$Z_k = Z_{CLT} = k_1 S_{gross} \quad (4.22)$$

S_{gross} is the section modulus for the complete rectangular cross-section of the CLT panel without considering the reduced section due to the transverse layers.

Shear analogy method

The final method presented is considered the most precise method as it does not neglect the effects of shear deformation (Blass & Fellmoser 2004). The shear analogy method splits the CLT panel into two virtual beams, A and B. Beam A is treated as the sum of flexural strength of the individual plies along their local neutral axis, while beam B accounts for the flexible shear strength of the panel and the flexibility of the connectors. Equation 4.23 calculates the true bending stiffness, where the values for BA and BB for the two virtual beams, are given in Equations 4.24 and 4.25, respectively.

$$EI_{eff} = B_A + B_B \tag{4.23}$$

$$B_A = \sum_{i=1}^{n} E_i I_i \tag{4.24}$$

$$B_B = \sum_{i=1}^{n} E_i A_i z_i^2 \tag{4.25}$$

The shear stiffness of the beam is considered for this method and is calculated using:

$$GA_{eff} = \frac{a^2}{\left[\left(\frac{t_1}{2G_1 b}\right) + \left(\sum_{i=2}^{n-1} \frac{t_i}{G_i b_i}\right) + \left(\frac{t_n}{2G_n b}\right)\right]} \tag{4.26}$$

Where:

$$a = t_{total} - \frac{t_1}{2} - \frac{t_n}{2} \tag{4.27}$$

t_i is the thickness of layer i

G_i is the shear modulus of layer i

b_i is the width of layer i

The method in the CLT Handbook by FPInnovations presents a simplified method to calculate the bending moment capacity where the section modulus is given by the following:

$$Z_{simp} = Z_{CLT} = \frac{(EI)_{eff}}{0.5 E_1 t_{tot}} \tag{4.28}$$

Design for in-plane loads

At the Graz University, the representative volume element, RVE, is proposed to calculate in-plane loads (Bogensperger, Moosbrugger & Silly 2010). The size of one RVE is dictated by the thickness of the CLT panel and the width of a single board plus half of the gap width on each side. The RVE is subjected to only in plane stresses (normal and shear) and therefore the stresses and strains are constant over the entire cross-section. If the thickness is equal for all layers the RVE can be further divided into a representative volume sub-element, RVSE (see Figure 23). An RVSE has the same square surface but with a thickness composed of half the board thickness on both sides of an adhesive layer acting as a plane of symmetry.

In manufacturing, it is common not to edge glue (narrow face) all CLT boards. Even with edge gluing, cracks can form due to swelling and shrinkage. This means that shear forces will be acting in different directions on adjacent planes and cause a torsional stress at the glued interface. Therefore, the RVSE is used to calculate both shear and torsional stresses. This method is only valid for constant layer thickness, therefore for layouts with various thicknesses and strength grades it is recommended to adopt load bearing and design models that are available for glued laminated timber.

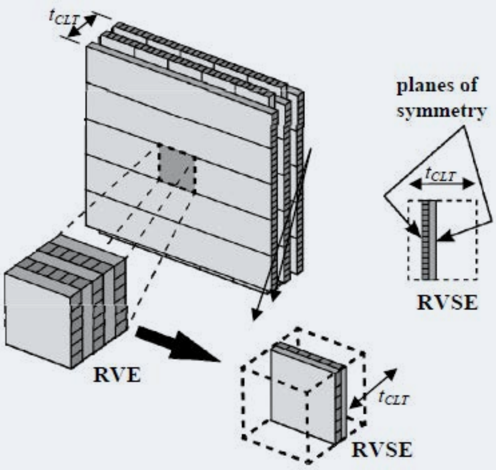


Figure 23: Definition of RVE and RVSE on a CLT element (Bogensperger, Moosbrugger & Silly 2010).

Earthquake design

Timber structures are lightweight and therefore the seismic actions are lower. However, the key to seismic design is structural ductility as it allows energy dissipation. Compared to the steel connections used to connect CLT panels, the panels themselves have infinite stiffness. The ductility of the structure therefore needs to be designed into the connections to ensure good seismic behaviour. Capacity-based design is proposed for seismic design as it aims to prevent brittle failure. By oversizing the CLT panels there is a global ductile failure mechanism at the connections (Gavric, Fragiacomo & Ceccotti 2015). Formulas provided by Eurocode 5 for connections with metal fasteners are used to ensure the connections are dissipative. In a CLT building these connections are the vertical screwed connections between wall panels, connections of wall to floor using angle brackets to resist shear and hold down connections at each end of a wall element to resist uplift.

Fire design

To check the strength capacity of the CLT during a fire, design is based on the reduced cross-sections per EN 1995-1-2 (2004). The charring depth depends on the adhesive applied, the gap size between boards and the availability of fire protection. The charring rate for softwoods and beech with density greater than 290 kg/m^3 is β = 0.65 mm/min for gaps up to 2 mm and β = 0.80 mm/min for gaps up to 6 mm. If the adhesive is not temperature proofed it has been observed that the charred layers of CLT elements loaded out of plane can detach.

Fire tests were conducted on CLT panels composed of various pine species in accordance with AS /NZS 3837:1998. The charring rate of pine with no gaps calculated from AS 1720.4-2006 is 0.75 mm/min for Hoop and Radiata and 0.64 mm/min for Slash pine. The experimental results on CLT panels with gaps displayed larger charring rates, close to the value of β = 0.80 mm/min provided by EN1995-1-2.

4.4.4 Serviceability Design

Short-term deflection

It is critical to calculate the deflection of CLT elements out-of-plane. Due to the cross-layers in CLT, the deformation due to the shear slip in the transverse layers is considered. The gamma method and the composite method incorporate this by using a reduction factor of the effective stiffness E_{Ieff}. The shear analogy method calculates the deflection due to shear slip.

Gamma method

The gamma method is straight forward to implement after the EI_{eff} has been calculated using Equation 4.15. The effective bending stiffness can then be used to calculate the deflection at any point. For the mid-span deflection under a uniformly distributed load it is calculated using:

$$\Delta_{mid,\gamma} = \frac{5wl^4}{384(EI)_{eff}} \tag{4.29}$$

Composite method

The composite method is straight forward to implement with the composition factor, k_1 calculated using Equation 4.21. The effective bending stiffness can then be used to calculate the deflection at any point. For the mid-span deflection under a uniformly distributed load it is given by:

$$\Delta_{mid,k} = \frac{5wl^4}{384k_1E_0I_{gross}} \tag{4.30}$$

Shear analogy method

The maximum deflection in the middle of a uniformly loaded CLT slab using the shear analogy method is given by Equation 4.31. The first term is the amount of deflection due to bending deformation, while the second term is the amount of deflection due to shear deformation.

$$\Delta_{mid,SAM} = \frac{5wl^4}{384(EI)_{eff}} + \frac{wl^2k}{8(GA)_{eff}} \tag{4.31}$$

k is a shear coefficient factor equal to 1.2 according to Timoshenko.

Long-term deflection

The factors for creep have been given by prEN 16351 (2011) and are dependent on the amount of moisture and relative humidity the structure is exposed to. The values proposed to be used are k_{def} = 0.85 and k_{def} = 1.1 for service class 1 and 2 respectively.

There is currently little information on how these values compare with CLT panels composed of Australian and New Zealand pine species.

4.5 Vibration

The design of CLT floors for vibration performance depends on three aspects of design: floor loads that cause the vibration response; response of the structure defined by the modal properties; and vibration perception/experience by the user measured using acceptability criteria (see Figure 24).

In regard to loading, the worst-case scenario – where the most problematic floors will have a resonant response due to a cyclic load, commonly walking – is considered. These floors will generally have a lower fundamental frequency. Annoyance in floor vibration can also occur due to an impact load. However, floors susceptible to impact load may not necessarily have a low fundamental frequency and the transient floor response needs to be computed for such floors. Once the loads are determined in step 1 of Figure 24, the modal properties in step 2 can be calculated. It is important to understand the loading as this can change the values of the natural frequencies and the damping. The modal properties can be calculated either by closed form solutions of beam or plate formulas for vibration or alternatively a finite element analysis can be used.

The acceptability of the floor can be determined by either a simplified prescriptive based method in step 3a or a more complex response factor analysis in step 3b. Generally, the prescriptive-based methods are for a limited floor types while the response factor analysis covers any floor type and loading case.

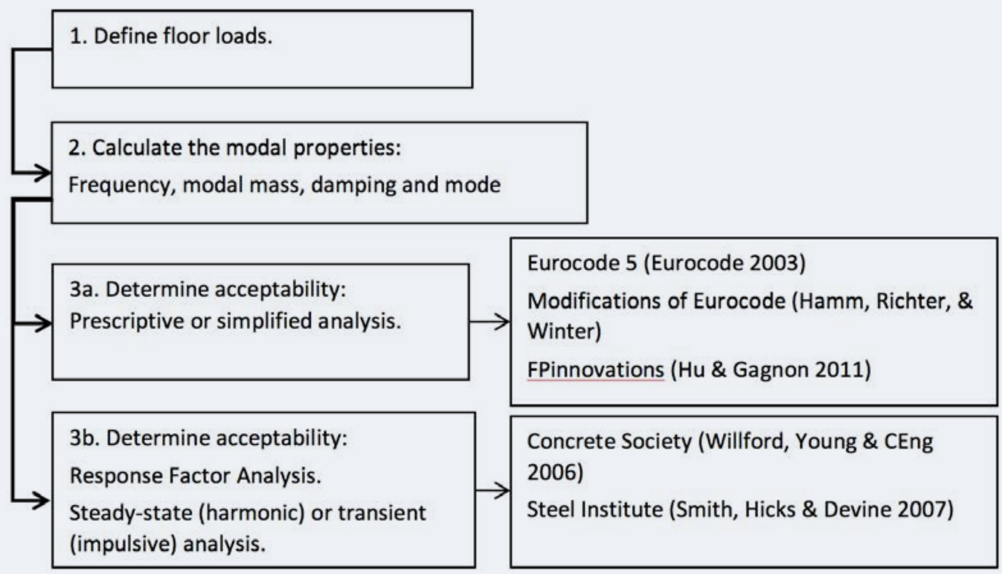


Figure 24: Summary of procedure to determine vibration performance of a floor.

4.5.1 Step 1 – Load Type

Walking loads are considered a cyclic load that can cause resonant frequency with a floor if the walking frequency is close to or equal to one of the natural frequencies of the floor. Generally, people walk with frequencies between 1.5–2.5 Hz. However, it is not as simple as avoiding floor designs with these low frequencies as up to the 4th harmonic of the walking load can excite a natural frequency – if a natural frequency is a multiple of 1-4 of the walking load, a resonant response can occur. Therefore, floors with the first natural frequency below 10 Hz (2.5Hz x 4) are considered resonant response floors while floors above 10Hz are considered to have a transient response.

4.5.2 Step 2 – Modal Properties

The modal properties can be calculated by simply using closed form solutions. However, the limitations of using these equations are that the boundary conditions are limited to the derived formula and the solutions are commonly based on beam theory. Cross-laminated timber is a plate-like element that is capable of spanning both one-way and two-way. While its flexural modes as a one-way spanning structure can be predicted using beam formulas, more advanced plate formulas are required for torsional modes and two-way spanning behaviour. This section contains formulas, where available, for predicting the modal properties of CLT. It also contains advice on predicting these properties using finite element analysis (FEA).

Frequency

The closed form solution for the natural frequency of a simply supported beam is given by Equation 4.32. The natural frequency is proportional to the ratio of the stiffness of the structure to the modal mass.

$$f_j = \frac{j^2\pi^2}{2\pi l^2}\sqrt{\frac{EI}{m}} \tag{4.32}$$

Where:

j is the mode number

l is the length

EI is the stiffness of the cross-section

m is the modal mass

This equation is limited to the Euler-Bernoulli theory of slender beams where the effects of shear deformation of the cross-section are assumed negligible. Due to the cross lamination of CLT, it is particularly susceptible to shear deformation. The slender beam formula can still, however, be adopted by finding an effective EI value that accounts for the loss of stiffness due to shear.

The formula can also be modified by multiplying it by a value of, K, to account for fixity type to become Equation 4.33. Values for K are given in Table 10.

$$f_j = \frac{K}{2\pi l^2}\sqrt{\frac{EI}{m}} \tag{4.33}$$

Table 10: Values for K, for beams with different end conditions (Willford, Young & CEng 2006).

End Condition	1st mode, K_1	2nd mode, K_2	3rd mode K_3
Pin – Pin	9.87	39.5	88.8
Fix – Free (Cantilever)	3.52	22.0	61.7
Fix – Pin	15.4	50.0	104
Fix – Fix	22.4	61.7	121

If the end fixity cannot be idealised as the examples in Table 10 and has some sort of partial restraint, the beam can be represented as a symmetrically elastically supported beam shown in Figure 25. Advanced computations are required to calculate the K values for this type of beam. Generalised solutions for this type of beam are given in (Karnovsky, Lebed & Karnovskii 2004).

Figure 25: Beam with symmetrically elastically restrained ends, (Wang & Wang 2013).

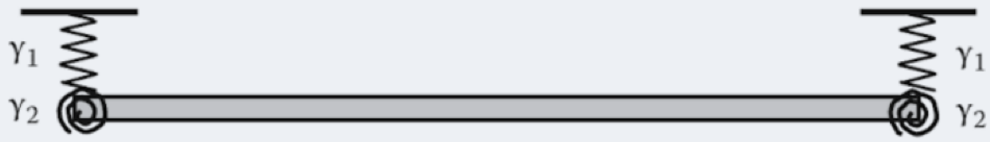


Figure 25: Beam with symmetrically elastically restrained ends (Wang & Wang 2013).

Beam theory, however, will only account for the flexural modes of vibration and will ignore the torsional and transverse modes present in the CLT plate structure. Experiments have indicated that the 2nd mode of a single simply supported CLT plate, which is a torsional mode, also has a large modal contribution factor and should also be considered for the design of CLT (see final project report, PNA 341-1415, available on the FWPA website). Table 12 shows the first five modes of vibration for a single panel of CLT, simply supported on each side,. Therefore, formulae for plate theory are required to capture these vibration modes. Unfortunately, this is not a trivial calculation and there are several textbooks, including Timoshenko Theory of Plates and Shells that are dedicated to solving closed form solutions for plate and shell structures (Timoshenko & Woinowsky-Krieger 1959).

The two-way spanning vibration modes can also be calculated using plate theory. Frequency for a simply supported plate with isotropic material properties can be calculated using Equation 4.34. In Equation 4.34, D is the flexural rigidity defined in Equation 4.35 and the dimensions of the plate are shown in Figure 26.

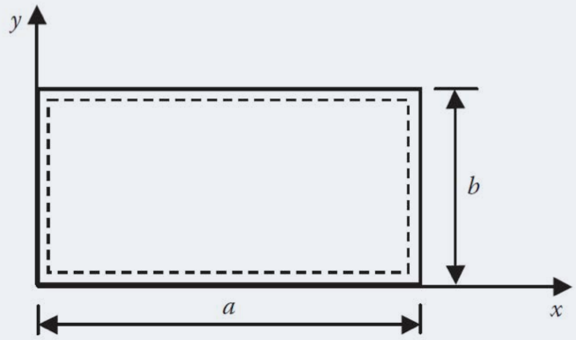


Figure 26: Coordinates and dimensions of two-way spanning plate.

$$f_{m,n,iso} = \frac{\pi}{2}\sqrt{\frac{D}{\rho t}}\left[\left(\frac{m}{a}\right)^2 + \left(\frac{n}{b}\right)^2\right] \tag{4.34}$$

Where:

ρ	is the density of the material
t	is the thickness of the plate
m	is the number of half sine waves in the x direction
n	is the number of half sine waves in the y direction
a,b	are the dimensions of the plate

$$D = \frac{Eh^3}{12(1-\upsilon^2)} \tag{4.35}$$

The isotropic equation can be modified to include the effects of the orthotropic nature of CLT. In each spanning direction, x and y, CLT has a strong and weak direction depending on the cross lamination. Equation 4.36 includes the orthotropic effect of CLT. Equation 4.37 gives the flexural and torsional rigidity.

$$f_{n,n,ortho} = \frac{\pi}{2a^2}\sqrt{\frac{m^4 D_x + 2Hm^2n^2\left(\frac{a^2}{b^2}\right) + n^4 D_y\left(\frac{a^4}{b^4}\right)}{m}} \tag{4.36}$$

Where:

m is the mass per unit area

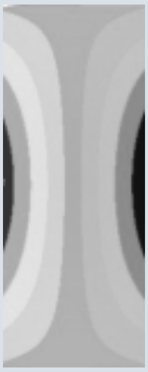
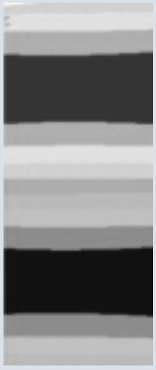
D_x, D_y are the flexural rigidity in the x and y direction

H is the torsional rigidity

$$D_x = D_y = H = \frac{Et^3}{12(1-v^2)} \tag{4.37}$$

However, these exact solutions are complex to derive and require significant computation. Finite element programs are becoming significantly easier to use and readily available. Therefore, in some cases it is more straightforward to perform FEA analysis to determine the frequency. The benefit of FEA is that mode shapes and modal masses are also easily extracted. Closed form equations discussed in this section can be used to check the validity of the FEA model.

Table 11: First five modes of vibration for a simply supported single 5.9 m long CLT panel.

Mode	1	2	3	4	5
Shape					
Frequency (Hz)	8.4	13.8	29.7	35.5	57.2

The theories discussed so far are for slender plates and beams. For some cases, using an effective EI value to capture the deformation is acceptable, but for span-to-depth ratios less than 15 and for concentrated loads these methods fail (Thiel 2013). Studies by Stürzenbecher et al. (2010) have compared various composite laminated advanced plate theories to produce a two-dimensional plate calculation. The advanced plate theories examined using Equivalent Single Layer Methods (ESLM), which means the number of independent variables is not dependent on the number of layers. ELSM is derived from 3D elasticity theory by making suitable assumptions concerning the stress state through the thickness of a laminate (Reddy 2006). These assumptions allow the reduction of the 3D problem to a 2D problem. The plate theory that was found to most accurately represent the plate behaviour of CLT was developed by (Ren 1986). The Ren theory contains a zigzag term that allows for discontinuous shear strains to represent the laminate specific characteristics. The theory by Ren (1986) was compared with a more accurate and time consuming exact solution by (Pagano & Hatfield 1972). It was found that the Ren plate theory closely matched the results of the exact solution without its computational difficulty.

Stürzenbecher et al. (2010) conducted further research to simplify this model by developing a 6-solution independent variable model (one less than Ren) that considers the relationship between the plane stress reduced stiffness components and the transverse shear stresses. It was found that compared to the Ren (1986) plate theory, the new theory delivers at least the same accuracy, and for transverse shear stresses even better, and at a lower computational cost. The extension of the presented plate theory to angle ply laminates and its implementation into finite element software is planned to make it applicable to structural simulations of plates with arbitrary lay-up, shape, and boundary conditions.

Modal Mass and Mode Shape

If performing finite element analysis, the modal mass can be extracted from a modal analysis. Care must be taken on whether a unity normalised or a mass normalised analysis is conducted. Most FEA packages allow you to choose. Either is okay to use if it is understood how each analysis affects the mode shape. If the structure is unity normalised, then the maximum displacement of the structure is set to 1 for every mode. The modal mass will then vary for each mode and should be used with the mode shape values for the unity normalised shape.

For a mass normalised analysis, the mode shape displacements are calculated from a modal mass of 1 kg for each mode. Therefore, a unity normalised measurement should occur if the modal masses are explicitly required. For example, in ANSYS, these can be extracted from the eigenvalue solutions or by converting the maximum kinetic energy for each mode into modal mass using Equation 4.38.

$$M_n = \frac{KE_n}{2\pi^2 f_n^2} \tag{4.38}$$

This equation can be checked by assessing that the first flexural mode is about half the weight of the floor.

Damping

In the CLT Handbook by FPInnovations values of damping ratios as low as 1% are used (Hu & Gagnon 2011). Tests found one-way spanning, simply supported CLT has a damping ratio for the first mode between 0.5% and 1.5%. Subsequent modes did not vary significantly. (See final project report, PNA 341-1415, available on the FWPA website.)

The damping is affected by the configuration, material, support conditions and the loading type. The occupant of a floor can change the damping characteristics. Onsite damping has been reported to be higher than in laboratory experiments. A study conducted in Sweden found that the damping ratio could be four times the value found in laboratory studies (Jarnerö, Brandt & Olsson 2015). Studies at UTS found laboratory CLT floors have damping ratios as low as 1%, while CLT floors tested in situ had damping ratios of 2.5% and 5% with no topping and with a 50 mm screed, respectively.

4.5.3 Prescriptive or Simplified Analysis

Prescriptive-based methods provide a simplified assessment of the vibration performance of a floor for a set of scenarios defined by the standard or design guide. These methods provide assessment of one or more of the following properties; stiffness, natural frequency, velocity, and acceleration of the floor. The methods generally contain equations to calculate the modal properties and give limits based on the type of floor. The methods compared here are from Eurocode 5 (2008), modifications of Hamm et al. (2010), modification by Mohr (1999) and the CLT Handbook criteria (Hu & Gagnon 2011). These methods and the criteria they use to assess the floor, including limit values are summarised in Table 12.

Table 12: Comparison of available analytical models for determining vibration performance.

	Stiffness (Unit Displacement)		Floor Natural Frequency		Floor Velocity		Acceleration (floors under 8 Hz only)	
Vibration Performance Method	Load kN	Limit mm	Load Case	Limit Hz	Velocity	Limit	Frequency Range Hz	Limit m/s2
Eurocode 5	1	≤1	G_{TOT}	≥8	Eq.(4.43)	Eq.(4.42)		
Hamm et al	2	≤0.5	G_{TOT}	≥8			4.5 – 8	≤0.05
Mohr	1	≤1	G_{TOT}+0.3Q	≥8	Eq.(4.47)	Eq.(4.47)	3.4 – 8	≤0.1
CLT Handbook	1	*	G_{TOT}	*				

* According to CLT Handbook criteria the floor frequency is dependent on the floor stiffness and vice versus.

The methods from European research and standards (Eurocode 5, Hamm et al. and Mohr) requires the vibration requirements of the floor to be defined first – either normal or high. High requirements are considered for commercial buildings and multi-storey residential blocks, whereas normal requirements are considered for single unit dwellings. Since this guide is concerned with long-span floors, primarily found in commercial buildings, high requirements for vibration are considered.

Eurocode 5

Eurocode 5 provides guidelines for providing acceptable vibration design of residential timber floors. Longitudinal stiffness (EI_l) and stiffness transverse to the span (EI_t), for a 1 m wide cross-section of CLT are used to calculate the natural frequency, deflection limit and floor velocity.

The natural frequency of the timber floor calculated using Equation 4.39, is limited to a minimum of 8 Hz, to avoid vibrations caused by resonance. Eurocode states that frequencies of 8 Hz can be acceptable with a 'special investigation' required; however, it does not provide guidelines for this investigation. The factor for support stiffness (k_m) in Eurocode 5 is equal to π^2 which represents a single span simply supported floor. For other end conditions the factors in Table 11 can be used.

$$f_1 = \frac{k_m}{2\pi l^2}\sqrt{\frac{(EI)_l}{m}} \geq 8\ Hz \tag{4.39}$$

The mass, m, is treated as a static mass; equal to the self-weight of the floor plus any extra imposed loads depending upon the use of the floor. Further to checking natural frequency, the deflection due to a unit force (Equation 4.40) is limited to a maximum value 'a', which is dependent on the required vibration performance level of the floor. A graph is provided in the code that displays the relationship between the limit value for deflection (a) and the limit value for velocity (b) (see Figure 27). The calculations are based on a rectangular floor supported on all four sides. Therefore, an equivalent beam width, b_{eff}, is calculated to determine the panel's equivalent beam effective stiffness (EI_b) taking into account the transverse stiffness using Equation 4.41 (Mohr, 1999).

$$w_{EC5} = \frac{1}{48}\frac{Fl^3}{EI_b} \leq a\, mm/k\,N$$

$$b_{eff} = \frac{l}{1.1}\sqrt[4]{\frac{EI_t}{EI_l}}$$

The velocity (v) due to an impulse of 1 Ns is then calculated using Equation 4.43 and limited by Equation 4.42. Only the number of first order modes with natural frequencies up to 40 Hz is considered and calculated using Equation 4.44. A value for damping, ζ = 1%, is provided by the code.

$$v \leq b^{(f_1\zeta-1)}\ m/(Ns^2)$$

$$v = \frac{4(0.4 + 0.6n_{40})}{mbl + 200}$$

$$n_{40} = \left\{\left(\left(\frac{40}{f_1}\right)^2 - 1\right)\left(\frac{b}{l}\right)^4 \frac{(EI)_l}{(EI)_b}\right\}^{0.25}$$

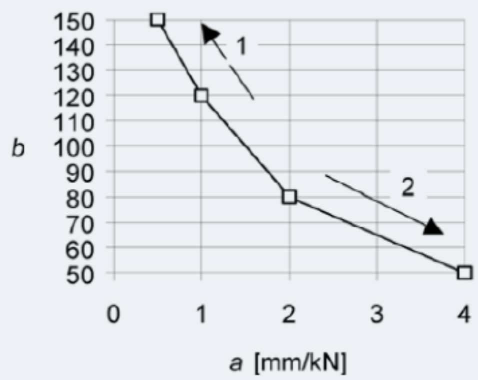


Figure 27: Interaction between the limit values of a, and b; directions 1 and 2 correspond to better and worse behaviour respectively (CEN 208).

Modifications of Hamm et al. (2010)

Modifications of the Eurocode 5 method were developed by Hamm et al. (2010) in Germany to account for the stricter requirements on vibration performance and for floors with natural frequencies less than 8 Hz. The research, which was based on the assessment of 50 buildings and 100 floors, found timber floors with natural frequencies less than 8 Hz, particularly heavy floors, could have acceptable vibration performance. A light floor on the other hand could perform poorly when subjected to frequencies over 8 Hz. Figure 28 is a flow chart that outlines the design procedure.

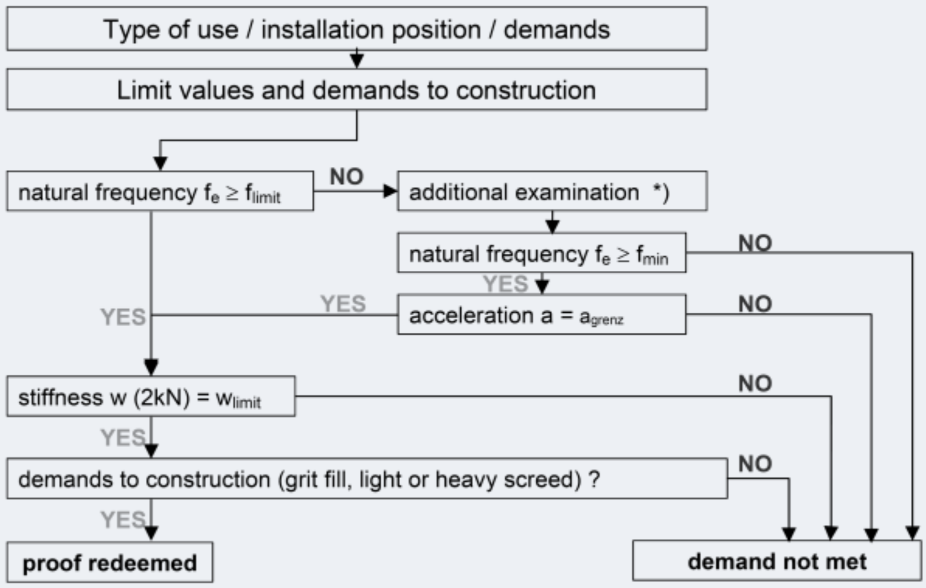


Figure 28: Flow chart for the design and construction of timber floors, the additional examination only applies for heavy floors with wide spans, or timber concrete composite systems (Hamm et al. 2010).

The frequency is calculated using the same method as Eurocode 5 considering only the static mass of the floor. The stiffness criterion is also calculated using a similar method as the Eurocode, however, it is given a more stringent limit value of 0.5 mm and a concentrated load value of 2 kN as opposed to 1 kN. The more stringent criteria were determined by studying the behaviour of several floors (Hamm et al. 2010).

If the frequency of the floor is less than 8 Hz, the floor is not necessarily deemed unacceptable, unlike the Eurocode. An additional examination of the acceleration is provided along with the original criteria also being met. The acceleration is calculated using Equation 4.45 and is limited to 0.05 m/s^2. In Equation 4.45, P_0 is the force of one person (taken as 700 N) and the values for the Fourier coefficient α_i and the forcing frequency FF are given in Table 13. The generalised mass, M_{gen}, is equivalent to half the effective area contributing to vibration performance (Equation 4.46) where the mass, m, is the self-weight of the floor plus any super-imposed dead load. Values for damping were taken as 1% as outlined by Eurocode 5.

$$a \approx 0.4 \frac{P_0 \alpha_i (f_1)}{M_{gen}} \frac{1}{\sqrt{\left[\left(\frac{f_1}{f_F}\right)^2 - 1\right]^2 + \left(2D\frac{f_1}{F_F}\right)^2}} \leq 0.05 \ m/s^2 \tag{4.45}$$

$$M_{gen} = m\frac{l}{2}b_{eff} \tag{4.46}$$

Table 13: Fourier coefficient, dependent on the fundamental frequency of the floor (Mohr, 1999).

Fundamental Frequency Hz	Fourier coefficient	Forcing frequency F_F Hz
3.4 < f1 ≤ 4.6	0.2	f_1
4.6 < f1 ≤ 5.1	0.2	f_1
5.1 < f1 ≤ 6.9	0.06	f_1
f_1 > 6.9	0.06	6.9

Mohr Criteria

The International Council for Building Research Studies and Documentation provides an alternate modification to the Hamm et al. (2010) method for frequencies below 8 Hz and was developed at the Technical University of Munich (Mohr 1999). This method considers a quasi-static floor mass that includes a portion of the live load in the total floor mass (G + 0.3Q) for calculating the natural frequency. Apart from the floor mass being quasi-static, both the frequency and the floor stiffness are calculated by the same method as Eurocode 5. A floor velocity check is included that was derived from the action of a 'heel drop' and is given by Equation 4.47. A damping value of 1% is assigned to floors without any additional boarding's for sound isolation as outlined by Mohr (1999).

$$v_{MOHR} = \frac{0.6}{m_f^{0.5} EI_l^{0.25} EI_t^{0.25}} < v_{lim,MOHR} = 6\times100^{(f\zeta-1)} \quad (4.47)$$

For floors with frequency below 8 Hz the acceleration is calculated using the same methods as outlined by Hamm et al. (2010). However, the acceleration limit is less stringent at 0.1 m/s^2.

CLT Handbook

A Canadian research team, FPInnovations, developed a simplified method to specifically assess the vibration performance for CLT floors, which was published in the CLT Handbook (Hu & Gagnon 2011). The criterion given by Equation 4.48 provides an inequality based on the fundamental frequency and the effective stiffness of the floor under a unit load.

$$\frac{f}{\Delta^{0.7}} \geq 13 \quad (4.48)$$

The deflection is calculated considering a 1 m-wide CLT panel and the frequency is calculated considering static mass only.

4.5.4 Response Factor Analysis

The prescriptive-based methods (discussed in the previous section) allow calculations of only certain floor types. Using response factor analysis, any floor type can be considered. Two design guides are considered here. The first was released by the Steel Construction Institute (Smith et al., 2009) and its scope is limited to steel framed floor and building types. The scope of the second guide, which was released by The Concrete Society Willford et al., 2006), is not limited to concrete structures and includes any other form of construction material that people walk on, including floors and bridges.

Both documents split the analysis up into resonant and transient response structures. Resonant response structures are defined by having any natural frequencies less than 10 Hz while impulsive or transient response structures have the first natural frequency above 10.5 Hz. This categorisation is based on the possible harmonics of walking frequencies that can influence a resonant response. If the structure's first natural frequency is around 10 Hz, both transient and resonant responses are advised to be analysed.

Concrete Society design guide

This section presents an example of a vibration response function procedure in accordance with the Concrete Society design guideline (Willford et al., 2006). The guideline provides an in-depth step-by-step procedure for determining the response function and should be consulted for analysis. A summary and example calculation of a resonant response structure is only discussed in this section as long-span floors have low fundamental frequencies and are likely to fall into the resonant zone.

The floor plate considered has been tested (final report for PNA 341-1415 is available on the FWPA website). The floor properties including material values are outlined in Appendix B.1. The floor consists of three CLT panels connected by half lap connections spanning between two timber support beams. Figure 29 shows the floor plan and the walking path used to activate the vibration response of the floor.

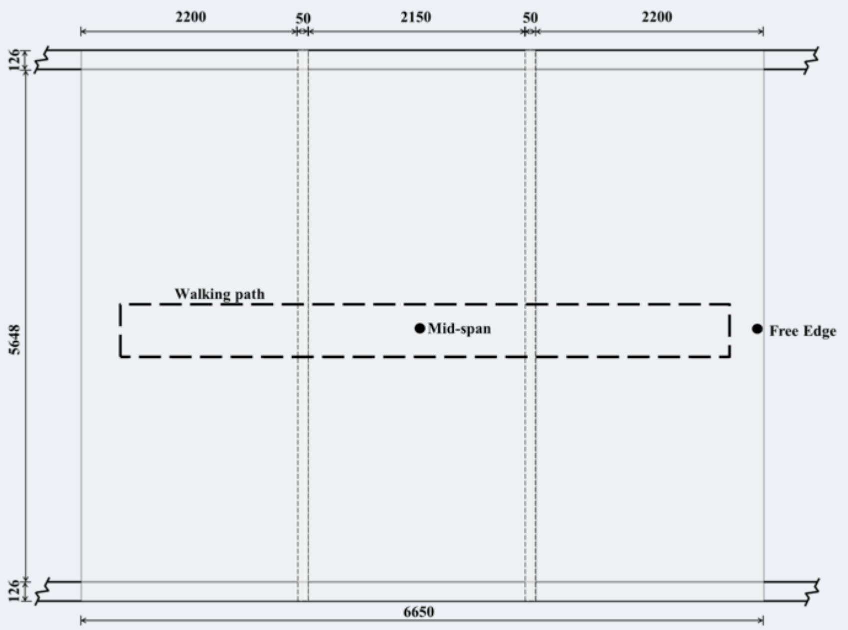
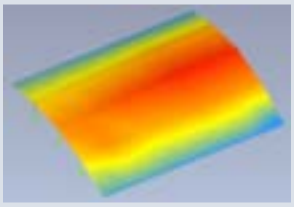
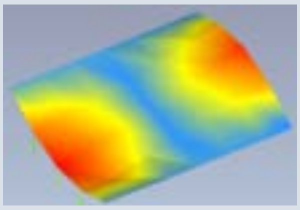
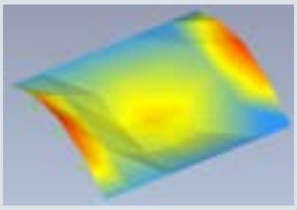
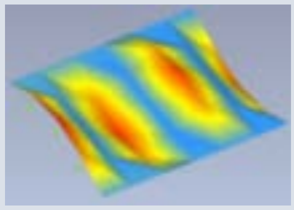


Figure 29: Floor dimensions of CLT floor plate and walking path.

The modal properties of the floor are first determined. This can be done by any appropriate method; closed form solutions, FEA and by experimentation. Table 14 shows the values of the first four natural frequencies, damping ratios and mode shapes, from walking tests conducted on the CLT floor plate.

Table 14: Results from ARTeMIS operational modal analysis software for the first four modes of vibration of the CLT floor plate.

Mode 1	Mode 2	Mode 3	Mode 4
f_1 = 7.72	f_2 = 9.74	f_3 = 13.86	f_4 = 19.15
ζ_1 = 1.2%	ζ_2 = 1.2%	ζ_3 = 1.1%	ζ_4 = 1.0%
			

The maximum floor response is going to occur due to walking loads that have a harmonic that can cause resonance with a natural frequency of a floor. The following process should occur not only for harmonics with the fundamental frequency but with any natural frequencies less than 10 Hz.

Step 1:

The first step is to calculate the harmonic forcing frequency, f_h. This is done by multiplying the walking frequency, f_w, by the harmonic number. For the floor tested the walking frequency was selected to be 2.57 Hz, which is listed as the 1st harmonic in Table 15. This walking frequency was selected as its 3rd harmonic corresponds with the fundamental frequency of the floor.

Step 2:

The forcing frequency, F_h, for each mode under 15 Hz is calculated. The guide contains the table to conduct this calculation.

Step 3:

The real and imaginary acceleration is then calculated and summed to find the magnitude of the response using the following equations:

$$a_{real,h,m} = \left(\frac{f_h}{f_m}\right)^2 \frac{F_h \mu_{r,m} \mu_{e,m} \rho_{h,m}}{m_m} \frac{A_m}{A_m^2 + B_m^2} \tag{4.49}$$

$$a_{imag,h,m} = \left(\frac{f_h}{f_m}\right)^2 \frac{F_h \mu_{r,m} \mu_{e,m} \rho_{h,m}}{m_m} \frac{B_m}{A_m^2 + B_m^2} \tag{4.50}$$

Where:

$A_m = 1 - \left(\frac{f_h}{f_m}\right)^2$ and $B_m = 2\zeta_m \frac{f_h}{f_m}$

Where:

$\mu_{r,m}$	is the mode amplitude at the response location
$\mu_{e,m}$	is the mode amplitude at the excitation location
f_m	is the mode frequency
ζ_m	is the mode damping ratio
m_m	is the modal mass
$\rho_{h,m}$	is a correction factor to account for the likelihood of resonant response being reached by the number of footfalls possible and the length of the floor

The real and imaginary accelerations are summed for each mode the magnitude of the acceleration response is found using Equations 4.53 and 4.54.

$$a_{real,h,} = \sum_m a_{real,h,m};\ a_{imag,h} = \sum_m a_{imag,h,m} \quad (4.51)$$

$$|a_h| = \sqrt{a_{real,h}^2 + a_{imag,h}^2} \quad (4.52)$$

This process was conducted for two locations of the floor (see Figure 29), at the mid-span of the floor and at its edge.

Step 4:

The response factors are calculated by dividing the magnitude of the acceleration response by the baseline peak acceleration for a response factor of 1. Table 15 gives the results from the guideline at the free edge and at the mid-span.

For a walking speed of 120 bpm or 2Hz we find a response factor of 48 at the mid-span and 56 at the free edge. When the pace is increased to 2.57 Hz which has a harmonic that coincides with resonance the RF increases to 250 at mid-span and 300 at the free edge.

Table 15: Response factor results from the Concrete Society design guideline.

Walking Frequency	RF mid-span	RF free-edge
2 Hz (120bpm)	48	56
2.57 Hz (resonance)	250	300

The CLT floor produced very high response factors both at mid-span and at the edge of the floor plate. This is a bare CLT floor under laboratory conditions and finishes and partitions in an actual floor would add additional damping to the system. An observation from this method is that the response factors are highly sensitive to changes in frequency, mass and damping values. This leads to the tendency to want to increase mass and damping to lower the response factor value. For example, doubling the damping from 1.2% to 2.4% reduces the mid-span response factor from 250 down to 120. Changing the mass of a system requires re-analysis of the floor to assess the resulting natural frequencies. Care should be taken with both approaches as an increased mass leads to lower natural frequencies and known damping values are currently poorly defined.

Steel construction institute design guide

The document provided by the Steel Construction Institute (Smith et al., 2009) provides a comprehensive vibration design assessment for steel-framed floors. While the document states that its scope is limited to steel structures, it provides a framework that can be used for any material type. The design process is dependent on the availability of finite element (FE) modelling. A simplified method is provided if FE analysis is not available.

The response analysis is based on the conservative assumption that the vibrating force will be applied to the most responsive part of the floor. This is a logical assumption given the most responsive location is generally the centre of a floor where movement will likely occur.

For low frequency floors (defined by Table 16) both a steady state response and a transient response are conducted. This is because the higher frequencies of a low frequency floor can cause the transient response to be greater than the steady-state response. For floors defined as high frequency, only a transient response is considered.

Table 16: Definition of low frequency floors provided by Smith, Hicks & Devine (2007).

Floor Type	Low to high frequency cut-off
General floors, open plan offices, etc	10 Hz
Enclosed spaces, e.g. operating theatre, residential	8 Hz
Staircases	12 Hz
Floors subject to rhythmic activities	24 Hz

Steady state response of floors (resonant response)

The natural frequencies up to 2 Hz higher than the cut-off frequency in Table 17 for a low frequency floor are examined. The weighted root mean square (RMS) acceleration of the floor for a single mode of acceleration is calculated using Equation 4.53.

$$a_{w,rms,e,r,n,h} = \mu_{e,n}\mu_{r,n}\frac{F_h}{M_n\sqrt{2}}D_{n,h}W_h \qquad (4.53)$$

Where:

e is the point of excitation

r is the response location

n is the mode number

h is the harmonic number

$\mu_{e,n}$ is the mode shape amplitude at the excitation point

$\mu_{r,n}$ is the mode shape amplitude at the response location

F_h is the excitation force for the hth harmonic

M_n is the modal mass

$D_{n,h}$ is the dynamic magnification factor for acceleration

W_h is the weighting factor for human perception of vibrations

A discussion of how to calculate modal mass and the relationship of modal mass to the mode shape amplitude are included in section 3.5.1. The dynamic magnification factor $D_{n,h}$, which is the ratio of the peak amplitude to the static amplitude is calculated using Equation 4.54.

$$D_{n,h} = \frac{h^2\beta_n^2}{\sqrt{(1-h^2\beta_n^2)^2+(2h\zeta\beta_n)^2}} \qquad (4.54)$$

Where:

β_n is the frequency ratio of the walking frequency f_p to the mode frequency f_n (f_p/f_n)

ζ is the damping ratio

A weighting factor, Wh, is included to account for the perception of vibration that will cause discomfort in different building uses. For example, perceivable floor vibration in a hospital will cause more discomfort to an occupant than in an office or residential building. The weighting factors for vertical floor vibrations are dependent on the natural frequency of the floor and included in the design guide.

The total response of the system is then calculated by adding the accelerations of each mode contributing to the vibration response. The document discusses three methods to perform the summation of the response acceleration:

- **Full time history**: The most accurate methods which yields both peak and RMS accelerations but is computationally intensive. Since RMS accelerations are only required some simpler methods is presented.
- **Sum of peaks (SoP)**: Provides a conservative calculation as it assumes all the components of the response peak at the same time and continue to peak at the same time, i.e. it will calculate the RMS acceleration as proportional to 1/√2 of the peak acceleration where in reality the RMS will actually be somewhat lower. Figure 30 shows the relationship of SoP acceleration to the actual accelerations.
- **Square-root sum of squares (SRSS)**: The recommended method to determine the RMS acceleration response that will produce the same RMS acceleration as a full time history summation is calculated using the following equation:

$$a_{w,rms,e,r} = \frac{1}{\sqrt{2}}\sqrt{\sum_{h=1}^{H}\left(\sum_{n=1}^{N}\left(\mu_{e,n}\mu_{r,n}\frac{F_h}{M_n}D_{n,h}W_h\right)\right)^2} \qquad (4.55)$$

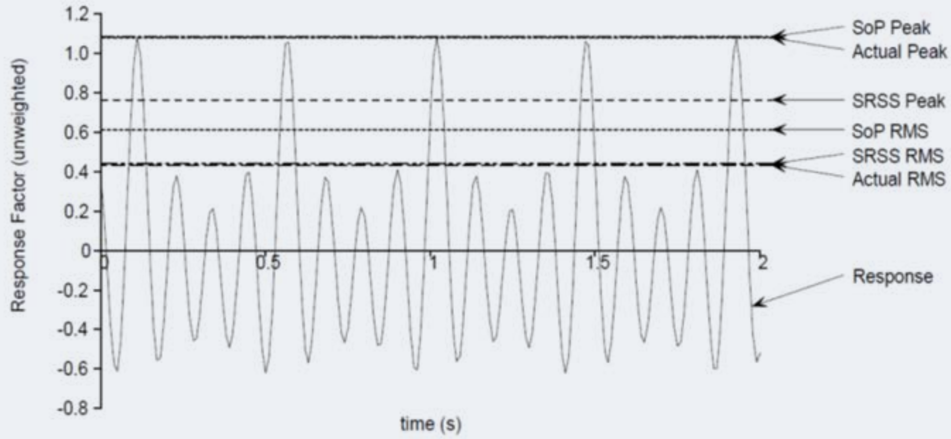


Figure 30: The relationship of peak and RMS accelerations for the time history vibration response of a floor.

The response factor is calculated by dividing the RMS acceleration with a base value acceleration of 0.005 m/s^2. The response factors for the CLT floor example are included in Table 18. At a walking speed of 120 bpm the mid-span response factor was found to be 51 while the free-edge was found to be 74. These values increased significantly to 270 and 390 when the walking speed corresponds to a floor harmonic.

Table 18: Response factor results from the SCI design guideline.

Walking Frequency	RF mid-span	RF free-edge
2 Hz (120 bpm)	51	74
2.57 Hz (resonance)	270	390

4.6 Experimental Observations

The CLT floor in Figure 29 was tested using two walking subjects. The walkers were instructed to walk at speeds of 60, 90 and 120 bpm and at a speed that corresponds to a harmonic of the fundamental frequency of the floor. The walkers took a number of different paths on the slab and would walk each path three times to ensure resonance build up was possible and that all areas of the floor were activated. Walker 1 had a weight of 52 kg while walker 2 was 65 kg. The accelerations were recorded, then weighted, filtered and the response factors calculated to compare with the predicted results.

4.6.1 The Effect of Using Extra Self-Tapping Screws at the Support

The size and number of screws at the support connection were varied and the response factors were recorded. The screws are partially threaded self-tapping screws with diameters of 6 mm and 8 mm and spacing from 125 mm up to 1000 mm. The cross-section of the span with the location of the self-tapping screw is shown in Figure 31.

There was a general trend for response to increase with increasing number of screws. The initial response from 0 screws to 6 screws at 1000 mm spacing was the most pronounced and then the change tapers off (see Figure 32). The maximum response factor was 14 for walker 1 and 18 for walker 2, this is an increase from 9.5 and 13 for having no screws at the support. Therefore, increasing the number of screws and hence the stiffness at the support has shown to have a negative influence on the vibration response of the floor.

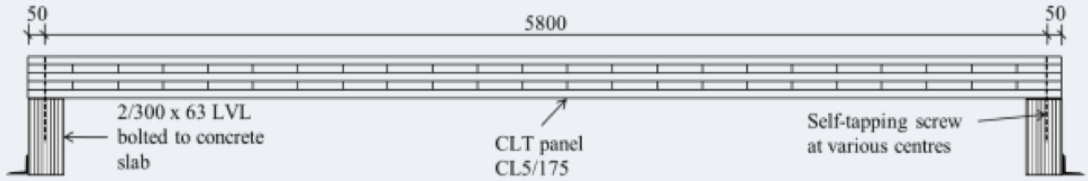


Figure 31: Cross section of single CLT panel connected to frame with a single screw connection.

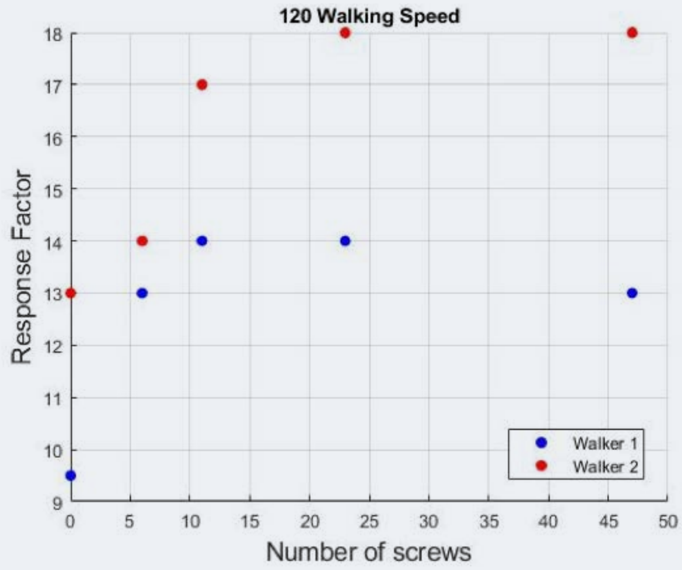


Figure 32: Response factors from walking speeds of 120 bpm on a single panel with increasing number of self-tapping screws at the support.

4.6.2 The Response of the Floor with 1, 2 and 3 Panels Adjacent to Each Other

Experiments were conducted on CLT floors spanning between two LVL beams. Three floor panels were tested with configurations of a single, double and triple panel floor (see Figure 33). The response factors were calculated over the entire floor of each configuration. However, for the purpose of this report the response factors are discussed at the centre of the span at both its mid-point and at the free edge (see Figure 33).

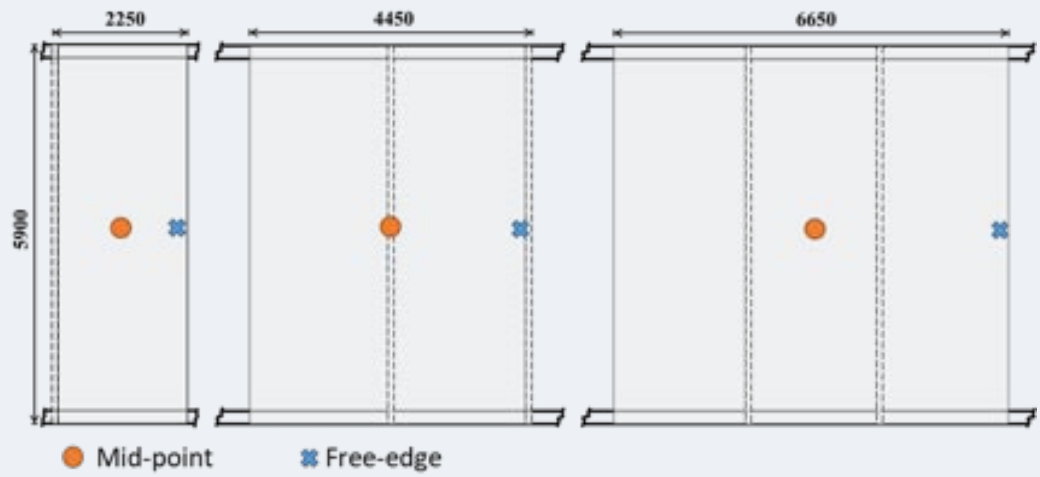


Figure 33: Layout of testing for CLT panels with locations of response factor calculations.

The response factor (RF) results from the walkers are recorded as a range in Table 18 at speeds of 60, 90 and 120 bpm. The response factor is dependent on the walker maintaining a steady and consistent walking pace. Because this is difficult to monitor, the range of response factors are recorded to indicate the variation in the data. It was found at the free edge, the response factors were relatively the same for each floor configuration. However, the response factors significantly decreased between the single panel and triple panel configuration at the mid-point of the floor. This is due to the extra mass of the three-panel floor compared to a single panel.

Table 18: Response factors of single, double and triple panel CLT floors.

	At free edge			At mid-point of floor		
Walking Speed	Single	Double	Triple	Single	Double	Triple
60	3.2–5.2	3.5–4.3	2.9–4	2.9–4.9	1.8–2.1	1.7–2
90	7.1–9.8	6–8.7	9.1–11	6.6–9	3.4–4.8	4.3–4.9
120	7.2–20	14–17	15–18	6.2–19	5.4–6.1	7.9–9.3

4.6.3 Added Support to the Free Edges of the CLT Floor

Generally, the largest vibration response occurs at the free edges of the CLT floor. Therefore, LVL support beams were added at the free edges to assess any improvement in the vibration response. By adding in the additional support, the CLT floor should theoretically act like a two-way plate (see Figure 34) as opposed to one-way spanning when there is no additional support.

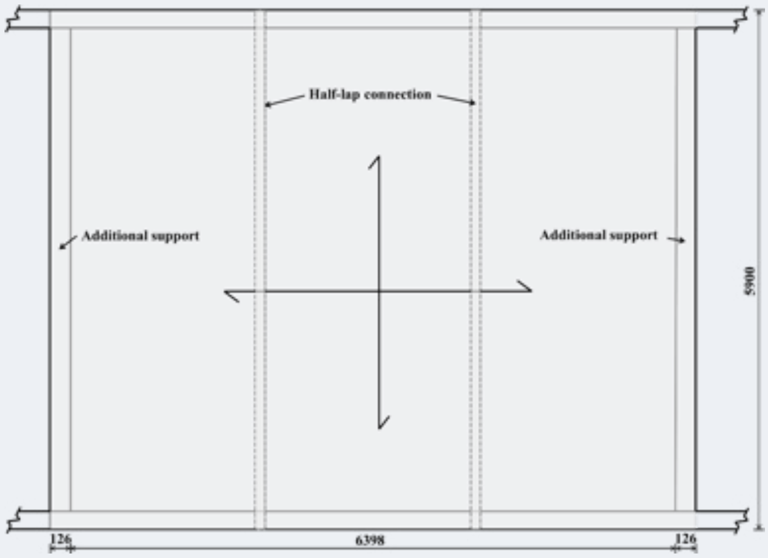


Figure 34: CLT floor with additional support at the free edges.

Table 19 gives the maximum response factors from the walkers and with the results from the one-way spanning floor. The maximum response factors have more than halved, from values of 15–18 for the one-way span down to 3.9–7.5 for the two-way floor at a walking speed of 120. Due to the extra support the maximum response factor of the two-way spanning floor is at the centre of the floor. The RFs at the centre of the two-way floor are smaller but not dissimilar to the RFs at the centre of the one-way spanning floor.

Table 19: Comparison of response factors of the one-way and two-way floor configurations.

	At free edge		**At mid-point of floor**	
Walking Speed	Max RF	Mid-point RF	Max RF	Mid-point RF
60	2.9–4	1.7–2	0.7–1.8	0.7–1.8
90	9.1–11	4.3–4.9	1.5–2.6	1.5–2.6
120	15–18	7.9–9.3	3.9–7.5	3.9–7.5

5 Discussion on Response Factor

Acceptable levels of vibration magnitudes are typically evaluated in industry as response factors (RF). The value indicates a level for which the probability of adverse comment is low. For commercial buildings, floors are mostly designed with a RF of less than 4, although this can be relaxed through consultations with the client.

The RF is proportional to the mass and damping of a structure. The higher the mass and damping ratio, the lower the response factor. Timber floors are light in weight and, therefore, produce high response factors that may not represent how they 'feel' in reality.

The floors tested at UTS (final project report, PNA 341-1415) were not designed to satisfy perception. These floors were built to examine their modal properties under various support and boundary conditions. However, it was noted by the staff and students working on each of the floors that the floors are significantly stiff and an examination of the RFs was then undertaken. The one-way spanning CLT system was found to have RFs up to 18 while the two-way spanning system values were up to 7.5. These are higher than the general accepted response factor of 4 for most systems. The ribbed deck cassette floor was also found to have higher RFs compared to accepted response factor (13 and 23 for pace frequencies of 2 and 2.14 Hz, respectively). However, procedure in CCIP-06, SCI P354 and DG 11 significantly overestimated the RF values for this type of floor, which was as high as 221% of the measured RF value.

On-site testing found that bare CLT floors produced RFs of about 30 while CLT floors with a concrete screed had RFs of about 10.

6 References

Australian Standard™, 2010, Timber Structures, Part 1: Design Methods, vol. AS 1720.1–2010, Standards Australia, Homebush (NSW), Australia.

Australian/New Zealand Standard™ 2002a, Structural Design Actions, Part 0: General Principles, vol. AS /NZS 1170.0:2002, Standards Australia, Homebush (NSW), Australia.

Australian/New Zealand Standard™ 2002b, Structural Design Actions, Part 1: Permanent, Imposed and Other Actions, vol. AS /NZS 1170.1:2002, Standards Australia, Homebush (NSW), Australia.

Allen, D. E., & Murray, T. M. (1993). Design criterion for vibrations due to walking. Engineering Journal, 4th quarter, 117–129.

ANSI, A. P. A. (2012). ANSI/APA PRG 320–2012 Standards for performance-rated cross-laminated timber. ANSI/APA, Tacoma, US.

AS . (2010). AS 1720.1 – 2010 Timber Structures Part 1: Design Methods. Australia: SAI Global.

Bernard, E. S. (2009). Improving Dynamic Behaviour in Lightweight Engineered Timber Floors. Australia.

Bishop, R. E. D., & Johnson, D. C. (1960). The analysis of real systems; approximate methods. In The Mechanics of Vibration (pp. 212–244). Cambridge, Great Britain: Cambridge University Press.

Blass, H. J., & Fellmoser, P. (2004). Design of solid wood panels with cross layers. In 8th world conference on timber engineering (Vol. 14, p. 2004).

Bogensperger, T., Moosbrugger, T., & Silly, G. (2010). Verification of CLT-plates under loads in plane. In 11th World Conference on Timber Engineering (WCTE2010), Riva del Garda.

British Standards Institute. BS 6472-1:2008 Guide to evaluation of human exposure to vibration in buildings, BS 6472-1:2008 (2008).

British Standards Institute. UK National Annex to Eurocode 5: Design of timber structures. General. Common rules and rules for buildings (2008).

Brownjohn, J., Racic, V., & Chen, J. (2016). Universal response spectrum procedure for predicting walking-induced floor vibration. Mechanical Systems and Signal Processing, 70–71, 741–755. https://doi.org/10.1016/j.ymssp.2015.09.010

CEN. EN 1995-1-1:2004 Eurocode 5: Design of timber structures – Part 1-1:General Common rules and rules for buildings (2004). The European Committee for Standardization. https://doi.org/10.1680/cien.2001.144.6.39

CEN. (2004b). Eurocoe 5: Design of Timber Structures.

Eurocode. (2003). Eurocode 5–Design of timber structures–Part 1-1: General rules and rules for buildings. prEN 1995-1-1. Bruxelles, Belgium.

Gagnon, S., & Pirvu, C. (2011). CLT handbook. Cross-Laminated Timber. FP Innovations Special Publication SP-528E.

Gagnon, S., & Popovski, M. (2011). Structural design of cross-laminated timber elements. Québec, FPInnovations.

Gavric, I., Fragiacomo, M., & Ceccotti, A. (2015). Cyclic behaviour of typical metal connectors for cross-laminated (CLT) structures. Materials and Structures, 48(6), 1841–1857.

Hamm, P., Richter, A., & Winter, S. (n.d.). Floor vibrations – new results.

HIVOSS. (2008). Vibration Design of Floors Guideline.

Hu, L., & Chui, Y.-H. (2004). Development of a design method to control vibrations induced by normal walking action in wood-based floors. In 8th World Conference on Timber Engineering. Lahti, Finland.

Hu, L. J., Chui, Y. H., & Onysko, D. M. (2001). Vibration serviceability of timber floors in residential construction. Progress in Structural Engineering and Materials, 3, 228–237. https://doi.org/10.1002/pse.69

Hu, L. J., & Gagnon, S. (2011). Vibration performance of cross-laminated timber floors. CLT Handbook. FPInnovations, Canada.

Jarnero, K. (2014). Vibrations in timber floors – dynamic properties and human perception. Linneaus University Dissertations.

Jarnerö, K., Brandt, A., & Olsson, A. (2015). Vibration properties of a timber floor assessed in laboratory and during construction. Engineering Structures, 82, 44–54. https://doi.org/10.1016/j.engstruct.2014.10.019

Karacabeyli, E., & Douglas, B. (2013). CLT handbook: cross-laminated timber. FPInnovations.

Karnovsky, I. A., Lebed, O., & Karnovskii, I. A. (2004). Free vibrations of beams and frames. McGraw-Hill Professional Publishing.

Labonnote, N. (2012). Damping in Timber Structures. Norwegian University of Science and Technology.

Mohr, B. (1999). Floor Vibrations. International Council for Building Research Studies and Documentation. Graz, Austria.

Ohlsson, S. V. (1988a). Springiness and human-induced floor vibrations: a design guide. Stockholm, Sweden: Swedish Council for Building Research.

Ohlsson, S. V. (1988b). Ten years of floor vibration research – a review of aspects and some results. In Symposium/Workshop on Serviceability of Buildings (pp. 435–450). Ottawa.

Ohlsson, S. V. (1991). Serviceability criteria – especially floor vibration criteria. In International Timber Engineering Conference (p. 1.58-1.65). London.

Pagano, N. J., & Hatfield, H. J. (1972). Elastic behavior of multilayered bidirectional composites. AIAA Journal, 10(7), 931–933.

Pavic, A., Reynolds, P., Prichard, S., & Lovell, M. (2003). Evaluation of Mathematical Models for Predicting Walking-Induced Vibrations of High-Frequency Floors. International Journal of Structural Stability and Dynamics, 3(1), 107–130. https://doi.org/10.1142/S0219455403000756

Reddy, J. N. (2006). Theory and analysis of elastic plates and shells. CRC press.

Ren, J. G. (1986). A new theory of laminated plate. Composites Science and Technology, 26(3), 225–239.

Smith, A. L., Hicks, S. J., & Devine, P. J. (2007). Design of floors for vibration: A new approach. Steel Construction Institute Ascot, Berkshire, UK.

Smith, A. L., Hicks, S. J., & Devine, P. J. (2009). Design of Floors for Vibration: A new approach. UK: The Steel Construction Institute.

Standard, A. (2010). AS 1720. 1—timber structures—part 1: design methods. Standards Australia.

Stürzenbecher, R., Hofstetter, K., & Eberhardsteiner, J. (2010). Structural design of Cross-laminated Timber (CLT) by advanced plate theories. Composites Science and Technology, 70(9), 1368–1379.

Taylor, S., & Hua, G. (2000). Dynamic performance of wood framed floor systems with poured toppings. In Sixth World Conference on Timber Engineering (p. 4.6.3.1-4.6.3.6.). Whistler.

TDA. (2015). Rethinking office construction – consider timber. Sydney, Australia: Timber Development Association NSW Ltd.

Thiel, A. (2013). ULS and SLS design of CLT and its implementation in the CLTdesigner. In Focus solid timber solutions—European conference on cross-laminated timber (CLT). The University of Bath, Bath.

Timoshenko, S. P., & Woinowsky-Krieger, S. (1959). Theory of plates and shells. McGraw-hill.

Toratti, T., & Talja, A. (2006). Classification of Human Induced Floor Vibrations. Building Acoustics, 13(3), 211–221. https://doi.org/10.1260/135101006778605370

Wang, C. Y., & Wang, C. M. (2013). Structural vibration: exact solutions for strings, membranes, beams, and plates. CRC Press.

Weckendorf, J., Zhang, B., Kermani, A., Reid, D., & Andersen, P. (2008). Damping Characteristics of Timber Flooring Systems with Respect to Low-Frequency Vibration Modes. In 10th World Conference on Timber Engineering. Miyazaki, Japan: Elsevier Ltd.

Willford, M. R., & Young, P. (2006). A design guide for footfall induced vibration of structures (CCIP-016). Surrey, UK.

Willford, M. R., Young, P., & CEng, Mim. (2006). A design guide for footfall induced vibration of structures. Concrete Society London, UK.

Willford, M., Young, P., & Field, C. (2007). Predicting footfall-induced vibration : Part 1. Proceedings of the Institution of Civil Engineers – Structures and Buildings, 160(SB2), 65–72.

Wyatt, T. A., & Dier, A. F. (1989). Floor Serviceability under Dynamic Loading. In International Symposium Building in Steel – The Way Ahead (p. 20.1-20.22). Stratford on Avon, UK: ECCS Publication No. 57.

Xiong, H., Kang, J., & Lu, X. (2011). Field Testing and Investigation of the Dynamic Performance and Comfort of Timber Floors. Journal of Asian Architecture and Building Engineering, 10(2), 407–412. https://doi.org/10.3130/jaabe.10.407

Zabihi, Z. (2014). Investigation of a Proposed Long Span Timber Floor for Non-Residential Applications. University of Technology Sydney.

A Appendix A. Worked example for a ribbed deck floor

A.1 Design of ribbed deck floor

This example demonstrates the ultimate and serviceability design of a ribbed deck cassette with top flange only. The flange and web are made from LVL 11 and LVL 13, respectively. The floor is located in a typical open plan commercial building. Vibration design as per procedures in CCIP-016, SCI P354 and DG 11 have also been undertaken including suggestions from this design guide.

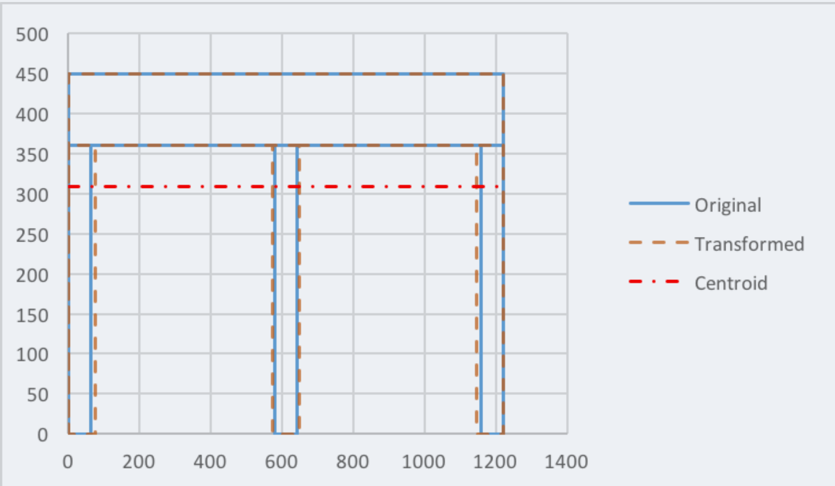


Figure A-1: Cross section of ribbed deck floor.

A.1.1 Floor structure

The floor cassette has a span of 9000 mm and consists of three 360 x 63 joists spaced equidistant apart. The cassette is 1220 mm wide. The cross-section can be seen in Figure A-1.

Cross section			
	depth	breadth	length
	mm	mm	mm
web	360	63	9000
top flange	90	1220	9000
bottom flange	0	0	0
total depth	450		
no. of joists/cassette	3		
no. of top panels	1		
no. of bottom panels	0		
Flange overhang	100	mm	

A.1.2 Material properties

The web and flange can be assumed to act compositely due to the glue and screw connection. The transformed section method has been used to convert the web to an equivalent width for the flange properties. The transformed cross-section can be seen in the dashed orange outline. The material properties have been taken from the manufacturer's technical data sheet. A density of 570 kg/m³ has been assumed.

Material properties								
	E	density	mass	f'b	f'c	f't	f's	f'p_{edge}
	MPa	kg/m3	kg/m	MPa	MPa	MPa	MPa	MPa
web	13200	569	12.9	48	38	33	5.3	10
top flange	11000	584	64.1	38	38	26	N/A	10
bottom flange	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

A.1.3 Section properties

This example demonstrates the ultimate and serviceability design of a ribbed deck cassette with top flange only. The flange and web are made from LVL 11 and LVL 13, respectively. The floor is located in a typical open plan commercial building. Vibration design as per procedures in CCIP-016, SCI P354 and DG 11 have also been undertaken including suggestions from this design guide.

Section properties

Element		A	y	Ay	Ixx	d	Ad2
		mm2	*mm*	*mm3*	*mm4*	*mm*	*mm4*
FLANGE	Top	109800	405	44469000	74115000	95.96	1E+09
FLANGE	Bottom	N/A	N/A	N/A	N/A	N/A	N/A
JOIST	1	27216	180	4898880	293932800	129.0	5E+08
JOIST	2	27216	180	4898880	293932800	129.0	5E+08
JOIST	3	27216	180	4898880	293932800	129.0	5E+08
JOIST	4	N/A	N/A	N/A	N/A	N/A	N/A
JOIST	5	N/A	N/A	N/A	N/A	N/A	N/A
JOIST	6	N/A	N/A	N/A	N/A	N/A	N/A
SUM		191448		59165640	955913400		2E+09

$\bar{y}$ =	309.0	mm
I=	3.327E+09	mm4
EI=	3.659E+13	Nmm2
Z_bot=	1.076E+07	*mm3*
Z_top=	2.360E+07	*mm3*
hc	141	*mm*
ht	309	mm

The web centre to centre spacing was checked for shear lag effects.

Cross section characteristics

Section 3.1 Wood Solutions Design Guide #30 Timber Cassette Floor System

Web c/c spacing	610	mm
If bottom flange exists c/c spacing - minimum of below		
bf,t	N/A	mm
bf,t	N/A	mm
If top flange exists, spacing to be less than number below		
bf,c ≤	963	mm
Check	*Satisfactory for shear lag effects*	

A.1.4 Load combinations and modification factors

Loading

G

	kg/m	kg	kN/m	
web	12.9	348.3	0.38	
top flange	64.1	703.8	0.77	
bottom flange	0	0	0	
		TOTAL G=	1.15	
		G=	116.9	kg/m
		G=	95.8	kg/m2

SDL

	kPa	kN/m	
services	0.25	0.305	
lighting	0.25	0.305	
floor finishes	1.5	1.83	
	TOTAL SDL=	2.44	
	G+SDL=	365.6	kg/m
	G+SDL=	299.7	kg/m2

LL

	kPa	kN/m	
commercial	3	3.66	
	TOTAL LL=	3.66	
	(G+SDL)+0.1LL=	402.9	kg/m
	(G+SDL)+0.1LL=	330.3	kg/m2

Load combinations

Ψ_s= 0.7

Ψ_l= 0.4

		w*	M*	V*	N*c	N*t
		kN/m	*kNm*	*kN*	*kN*	*kN*
ULS	**1.35G**	4.8	49.0	21.8	159.9	N/A
	1.2G+1.5Ψ_lQ	6.5	65.8	29.3	214.7	N/A
	1.2G+1.5Q	9.8	99.2	44.1	323.4	N/A

			w*	
SLS	**Short term**	**G+ Ψ_sQ**	6.15	kN/m
		1Kn deflection	1	kN/m
	Long term	**G+Ψ_lQ**	5.1	kN/m
		G only	3.6	kN/m

Modification factors							
k4, k6, k9, k12 factors							
Category	2						
Material	Structural LVL						
Lay	355 mm						
Force	Φ	k_4	k_6	k_9	k_{12}	k_7	
Tension	0.9	1	1	-	-	-	
Compression	0.9	1	1	-	1	-	
Shear	0.9	1	1	-	-	-	
Bending	0.9	1	1	1	1	-	
Bearing	-	-	-	-	-	1	

k12 calculation - stability factor

Cl 8.4.7		
	r	0.25
	pb	1.08
	S1	0
Check of continuous lateral restraint		
	LHS	0.99
	RHS	1.68
Continuous lateral restraint system		

k1 modification factors

Table G1

Load type	Combination	k_1	Comment
Permanent	1.35G	0.57	50+ years
Long-term	1.2G+1.5Ψ$_l$Q	0.8	5 months
Short-term	1.2G+1.5Q	0.94	5 days

A.1.5 Flexural design capacities

Flexural design capacities for the various loading conditions were checked using the *WoodSolutions Technical Design Guide 31: Timber Cassette Floors*. As shown in the utilisation table, the design is well below capacity.

Flexural design capacities

Combination	Bending		Axial		Shear	Bearing	
	Above centroid	*Below centroid*	*Top flange*	*Bottom flange*	*web*	*top flange*	*Bottom flange*
	Mdtop	Mdbot	Ndtop	Ndbot	Vd	Nptop	Nptop
	kNm	kNm	kN	kN	kN	kN	kN
1.35G	460.1	265.1	2140.4	N/A	347.0	625.9	N/A
1.2G+1.5Ψ$_l$Q	645.7	372.0	3004.1	N/A	487.0	878.4	N/A
1.2G+1.5Q	758.7	437.1	3529.9	N/A	572.3	1032.1	N/A

Utilisation

Combination	Bending		Axial		Bending and Compression		Bending and tension		Shear	Bearing
	Above centroid	*Below centroid*	*Top flange*	*Bottom flange*	*About xx*	*About yy*	*Eq 3.5(3)*	*Eq 3.5(4)*	*web*	*flange*
1.35G	0.11	0.18	0.07	N/A	0.18	N/A	N/A	N/A	0.06	0.03
1.2G+1.5Ψ$_l$Q	0.10	0.18	0.07	N/A	0.17	N/A	N/A	N/A	0.06	0.03
1.2G+1.5Q	0.13	0.23	0.09	N/A	0.22	N/A	N/A	N/A	0.08	0.04

A.1.6 Serviceability – deflection

Since the moisture content of the timber cassette is less than 15%, the creep factor, j_2, is equal to 1 and 2 for the short-term and long-term serviceability check, respectively.

Minimum required EI_{ef} for short-term and long-term deflection was less than the EI_{ef} of the section (3.659×10^{13} Nmm^2) and therefore satisfied the requirement. Deflection limits for short-term and long-term deflection are span/300 and span/400, respectively. This has been determined in accordance with Guidelines presented in Appendix B of AS 1720.1.

Serviceability - Deflection

**Short term and long term EI should be less than EI effective*

		Short term	Long term	1kN point load
EI_{ef}	*Nmm2*	1.75E+13	2.88E+13	
Δ mm		14.4	11.8	0.4
Δlimit mm		30	22.5	2
CHECK		OK	OK	OK

j2 factor - creep

Loading	j2
Instantaneous live load	1
Long-term loads in a controlled environment	2
Long-term loads in a variable environment	3

A.1.7 Serviceability – vibration design

Vibration performance was calculated using procedures in CCIP-016, SCI P354 and DG 11 to highlight differences between guides and assessed based on the Response Factor. Comparison of predicted to measured results of a ribbed deck cassette with dimensions as shown in Figure A-1 have been presented. The support condition is assumed to be the overhanging portion of the flange secured to the primary beam at 40 mm from the edge and is numerically represented as a pin-pin condition. The damping ratio is taken as 1%. Although in a design situation, the loading would include 10% live load as well as any other superimposed dead load, the loading condition for this assessment only considered the self-weight of the floor. With the aforementioned details, a finite element model was created with a mesh size of 20 mm. The modal properties of the first two modes obtained from a modal analysis are shown in Table 20 (other modes were greater than 30 Hz). Mode shape amplitudes for both modes were also extracted. Excitation and response locations were taken at mid-span along the centre joist. From the mode shapes shown in Figure 6(a) and (b), this means that there should be minimal contribution from mode 2. The fundamental frequency is 10.67 Hz which is above the cut-off frequency of all guides and thus a transient analysis assuming a 76 kg person has been undertaken.

Table 20: Measured modal properties from impact hammer testing

	Mode 1 (Bending 1)	**Mode 2 (Torsion 1)**
f_n Hz	10.67	11.48
m kg	464.3	87.6
ζ%	1	1

A.1.7.1 CCIP-016

For CCIP-016, the cut-off frequency between low and high frequency floors is determined using the formula 4.2 ×maximum walking frequency. The maximum walking frequency has been taken as 2.5 Hz (as recommended in CCIP-016) which results in a cut-off frequency is 10.5Hz. As such, this section follows the transient analysis procedure. The guide states that it is only necessary to check the response based on the maximum expected walking frequency since faster walking speeds induce greater responses. The modal superposition method is used to calculate the total response from both modes. As shown below, the predicted response factor considering the contribution of both modes is 124. The predicted velocity response for the period of one footstep (0.4 s) is shown in Figure A-2 for both modes where 'Mx' refers to Mode 'x'. The root-mean-square (RMS) values of each mode and total response from both modes (SUM) are also shown.

TRANSIENT ANALYSIS

	Node	Mode 1	Mode2
Exc node	8814	0.044911	0.013868
Resp Node	8814	0.044911	0.013868

		Mode 1	Mode 2	SUM
Walking frequency (fastest)	Hz	2.5	2.5	
Effective impulse	Ns	9.417	8.649	
Peak velocity	m/s	0.0189936	0.00166	
Period of footfall, T	sec	0.400	0.400	
Average of $v_m(t)^2$ over T	m/s	0.0001399	0.0000010	0.0001536
RMS velocity	m/s	0.011829	0.001018	0.012393
v,R=1	m/s	0.0001	0.0001	0.0001
Response Factor		118.3	10.2	123.9

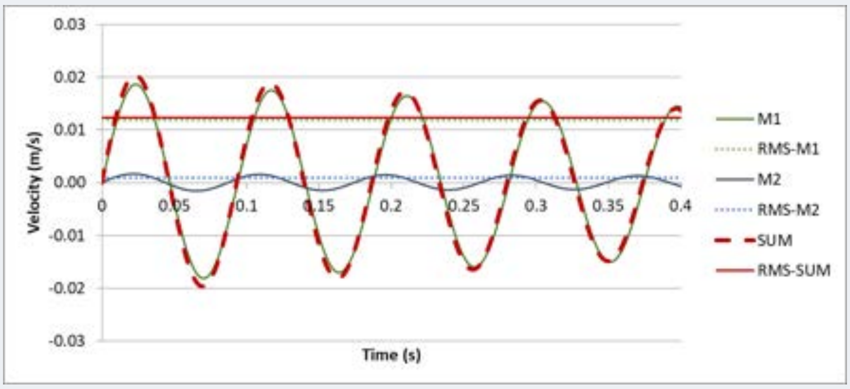


Figure A-2: Predicted velocity response for one footstep based on CCIP-016 procedure.

A.1.7.2 SCI P354

The cut-off frequency recommended for general floors and open plan offices is 10 Hz and subsequently a transient analysis according to the 'general assessment' procedure (Section 6 of SCI P354) is followed. The effective impulse equation provided in SCI P354 is expressed in conjunction with requirements provided in EN 1990 Annex C (Gulvanessian 2001; Smith, Hicks & Devine 2009). As a result for a 76 kg person the impulse applied is approx 18% higher than the design effective impulse in CCIP-016 and therefore a higher predicted response is expected. Despite SCI P354 recommending a maximum design pace frequency of 2.2 Hz, a pace frequency of 2.5 Hz is selected to compare with CCIP-016 results. As shown below, the predicted response factors for both modes is 199. The predicted acceleration response for one footstep as per SCI P354 approach is shown in Figure A-3.

TRANSIENT ANALYSIS				
	Node	Mode 1	Mode2	
Exc node	8814	0.044911	0.013868	
Resp Node	8814	0.044911	0.013868	
		Mode 1	Mode 2	SUM
Wn (Wb weighting)		1	1	
Walking frequency	Hz	2.5	2.5	
F_i	Ns	11.437	10.505	
Peak Acceleration	m/s2	1.522	0.142	
Period of footfall, T	sec	0.400	0.400	
Average of a,w(t)^2	m/s	0.898	0.008	0.992
RMS accel	m/s2	0.948	0.087	0.996
accel,R=1	m/s2	0.005	0.005	0.005
Response Factor		189.6	17.4	199.2

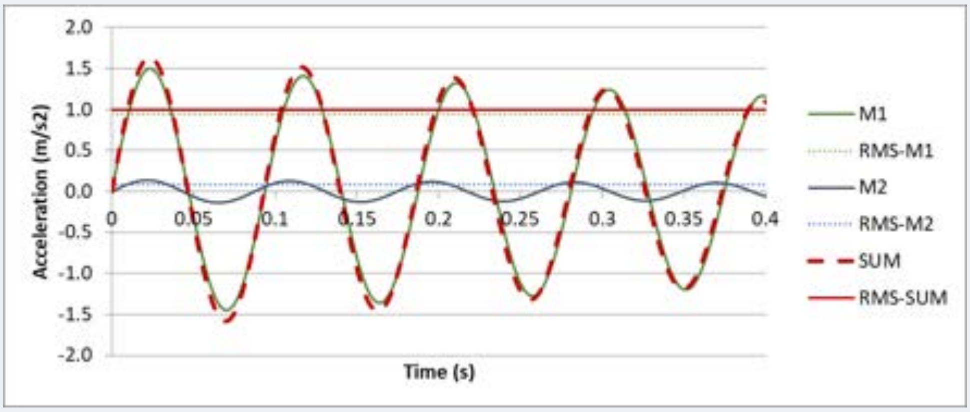


Figure A-3: Predicted acceleration response for one footstep based on SCI P354 procedure.

A.1.7.3 AISC DG 11

In AISC DG11 (2016), the cut-off frequency is taken as 9 Hz which means that, similarly to CCIP-016 and SCI P354, the response prediction follows a transient analysis using the finite element method (Section 7 in the guide). In a typical design scenario, the effective impulse would be calculated based on a 5th to 9th integer division of the dominant mode. However, in this case to allow direct comparison to predictions from other guides, a pace frequency of 2.5 Hz was used. As shown below, the total predicted response factor is 130. The predicted acceleration response for one footstep is shown in Figure A-4.

TRANSIENT ANALYSIS (from Section 7.4)				
	Node	Mode 1	Mode2	
Exc node	8814	0.044911	0.013868	
Resp Node	8814	0.044911	0.013868	
		Mode 1	Mode 2	SUM
Walking frequency	Hz	2.5	2.5	
I_eff	Ns	7.338	6.675	
Peak Acceleration (mass normalised)	m/s2	0.99263	0.09260	
Period of footfall, T	sec	0.4	0.4	
Average of a,w(t)²	m/s	0.3821861	0.0032127	0.4221163
RMS accel	m/s2	0.618212	0.056680	0.649705
Accel_ESPA	m/s2	0.874283828	0.080158	0.9188213
	%g	8.91%	0.82%	9.37%
accel,R=1	m/s2	0.005	0.005	0.005
Response Factor		123.6	11.3	129.9

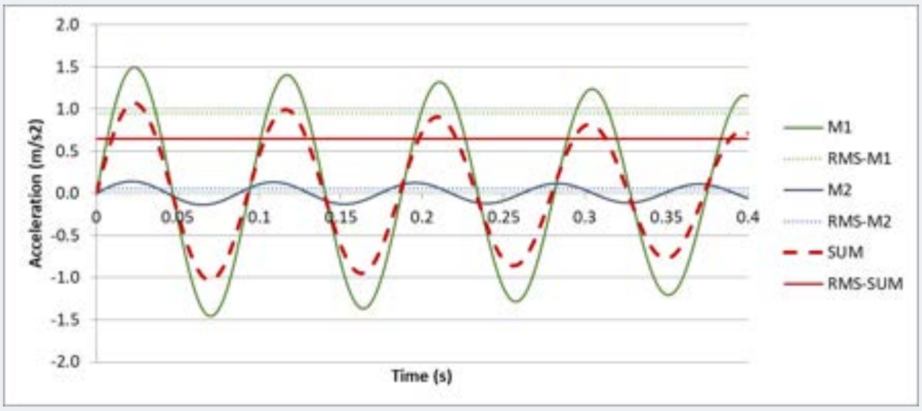


Figure A-4: Predicted acceleration response for one footstep based on AISC DG11 procedure.

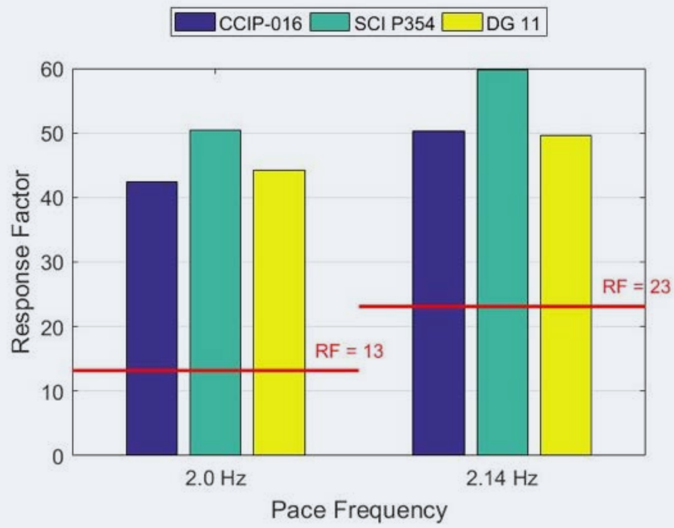


Figure A-5: Comparison of predicted Response Factors based on CCIP-016, SCI P354 and DG 11.

B Appendix B. Worked example for a Cross-laminated Timber Floor

B.1 Floor Properties

This section gives the geometry and material properties of a CLT floor that is used to demonstrate the design procedures required for a CLT floor.

The floor consists of three 2.2 m wide panels spanning 5.8 m, making a 5.8 x 6.6 m^2 floor plate. The example calculations will discuss the design for the panel supported by primary beams in a one-way span arrangement and also supported on all four sides, thereby spanning two-ways. The floor geometry is summarised as:

Floor span, L = 5.8 m
Panel width, w = 2.2 m
Panel thicknesst_{tot} = 175 mm
Number of panels, n = 3

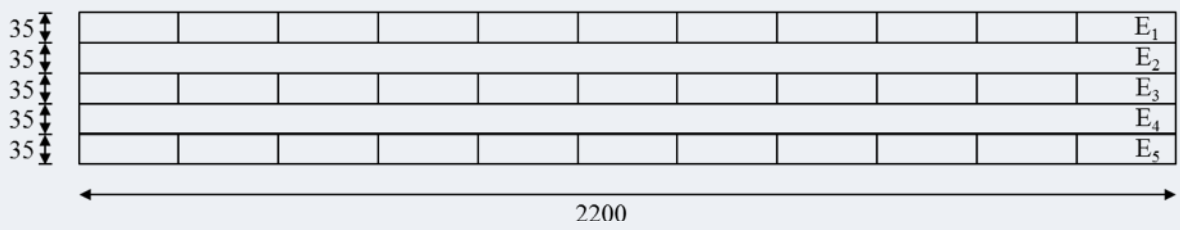


Figure B-1: Cross section geometry of 5 layered CLT panel.

The material used to compose the panel is New Zealand Radiata Pine. The cross-section shown in Figure B-1 is made up of five equal thickness layers of timber. The characteristic strength properties required to complete the design process in this section is included in Table 21.

Table 21: Material properties of CLT panel for example calculation.

Layer number	Thickness (mm)	Elastic Modulus (MPa)	Bending Strength (MPa)	Tension Strength (MPa)
1	35	8000	14	6

The density is 460 kg/m^3.

B.2 Strength Design

This design guideline covers several methods and numerous calculations are involved for each method. This section contains the results from the example floor and demonstrates the abilities of each method.

Table 22: Material properties of CLT panel for example calculation.

Property	CLT Designer	Gamma	Composite	Shear Analogy
$Z_{eff,1m}$ mm^3	4.04 x 10^6	3.99 x 10^6	4.06 x 10^6	4.09 x 10^6
Bending Moment, kNm	36.6	36.2	36.8	37.1
$A_{eff,1m}$ mm^2	136 x 10^3 (mid) 144 x 10^3 (rolling)	133 x 10^3 (mid) 189 x 10^3 (rolling)		
Shear strength, kN	55	72	-	-
Bearing Strength, kN	637	-	-	-

B.2.1 CLT Designer

Using Equation 4.4 to calculate the floors bending stiffness, we find:

$$K_{CLT} = (2\times3.573\times10^6\times8000 + 3.573\times10^6\times8000) + (2\times8000\times35000\times70^2)$$
$$= 2.83\times10^{12} Nmm^2$$

And therefore, the section modulus:

$$Z_{CLT} = \frac{2\times2.83\times10^{12}}{175\times8000} = 4.04\times10^6\ mm^3$$

The bending strength is given by:

$$Z_{CLT} = \frac{2\times2.83\times10^{12}}{175\times8000} = 4.04\times10^6\ mm^3$$

Using the equation for moment capacity in accordance with AS 1720.1 the ultimate moment capacity for the CLT is calculated under permanent loads:

$$M_d = 0.95\times0.57\times1.33\times12.6\times4.04\times10^6 = 36.6\ kNm$$

The shear stress requires calculation of the effective area using (4.13) for the mid-span and (4.14) for the rolling shear layers:

$$A_{eff,mid} = \frac{2.83\times10^{12}}{\left(8000\times35\times70 + \frac{8000\times35^2}{8}\right)} = 136\times10^3\ mm^2$$

$$A_{eff,rolling} = \frac{2.83\times10^{12}}{8000\times35\times70} = 144\times10^3\ mm^2$$

The shear strength at mid-section, $f_{v,CLT,d}$ and rolling shear strength $f_{r,CLT,d}$ of the timber are 3.0 N/mm^2 and 0.7 N/mm respectively (Unterwieser & Schickhofer, 2014).

The shear strength is then the minimum of the shear at mid-span or the shear at the transverse layers:

$$V_{mid} = 0.95\times0.57\times3\times136\times10^3 = 221\ kN$$

$$V_{rolling} = 0.95\times0.57\times0.7\times144\times10^3 = 55\ kN$$

The bearing area is a line load along the support of the CLT panel with a support area of 125 mm x 2200 mm. Therefore, the bearing capacity is:

$N_{d,p}$ =0.95×0.57× 1.5 ×2.85 ×125×2200= 637 kN

B.2.2 Gamma Method

Using Equation 4.16 we find the gamma reduction value:

$$\gamma_1 = \frac{1}{1+\pi^2\frac{8000\times35000}{5800^2}\cdot\frac{35}{50\times1000}} = 0.95$$

Where the rolling shear modulus is taken as 50 MPa.

$\gamma_1 = \gamma_3 = 0.95$ and $\gamma_2 = 1$

$z_1 = z_3 = 70$ mm and $z_2 = 0$

And the effective stiffness is found to be:

$$EI_{eff} = 2.68{\times}10^{12}\ Nmm^2$$

$$Z_\gamma = \frac{(EI)_{eff}}{E_1(\gamma_1 z_1 + 0.5t_1)} = \frac{2.68{\times}10^{12}}{8000(0.95{\times}70 + 0.5{\times}35)} = 3.99{\times}10^6\ \text{mm}^3$$

Finally the bending moment capacity is calculated in accordance with AS 1720.1:

$$M_d = 0.95{\times}0.57{\times}1.33{\times}12.6{\times}3.99{\times}10^6 = 36.2\ \text{kNm}$$

The effective shear area is calculated using:

$$A_{eff,mid} = \frac{(EI_{eff})b}{\left(\gamma_1 E_1 A_1 z_1 + E_1' A_1' z_1' + \gamma_2 E_2 \frac{A_2}{2}\frac{t_2}{4}\right)} = \frac{2.68{\times}10^{12}\ {\times}1000}{\left(0.95{\times}8000{\times}35000{\times}70{+}267{\times}35000{\times}35{+}1{\times}8000{\times}\frac{35000{\times}35}{2\quad\ 4}\right)} = 133{\times}10^3\ \text{mm}^2$$

E_1' is taken as $E_1/30$ = 267 MPa

z_1' = 35 is the distance from centroid of rolling shear layer and the centroid of the cross-section.

$$A_{eff,rolling} = \frac{(EI_{eff})b}{\left(\gamma_1 E_1 A_1\left(z_1 - \frac{t_2}{2}\right) + E_1' A_1'\left(z_1' - \frac{t_2}{2}\right)\right)} = \frac{2.68{\times}10^{12}\ {\times}1000}{\left(0.95{\times}8000{\times}35000\left(70 - \frac{35}{2}\right) + 267{\times}35000\left(35 - \frac{35}{2}\right)\right)} = 189{\times}10^3\ \text{mm}^2$$

The shear strength is then the minimum of the shear at mid-span or the shear at the transverse layers:

$$V_{mid} = 0.95{\times}0.57{\times}3{\times}133{\times}10^3 = 216\ \text{kN}$$

$$V_{rolling} = 0.95{\times}0.57{\times}0.7{\times}189{\times}10^3 = 72\ \text{kN}$$

B.2.3 Composite K-Method

The composite factor is calculated:

$$k_1 = 1 - \left[\left(1 - \frac{200}{8000}\right)\left(\frac{105^3 - 35^3}{175^3}\right)\right] = 0.797$$

$$EI_{eff} = 0.797{\times}8000{\times}\frac{1000{\times}175^3}{12} = 2.85{\times}10^{12} mm^4$$

$$Z_k = 0.797{\times}\frac{1000{\times}175^2}{6} = 4.06{\times}10^6\ \text{mm}^3$$

$$M_d = 0.95{\times}0.57{\times}1.33{\times}12.6{\times}4.06{\times}10^6 = 36.8\ \text{kNm}$$

B.2.4 Shear Analogy Method

The elastic modulus, shear modulus and rolling shear modulus for this calculation are taken as:

$E_{90} = E_0/30 = 8000/30 = 267$ MPa
$G_0 = E_0/16 = 8000/16 = 500$ MPa
$G_{90} = G_0/10 = 500/10 = 50$ MPa

$$B_A = 2\times 8000\times 3.573\times 10^6 + 8000\times 3.573\times 10^6 + 2\times 267\times 3.573\times 10^6 = 87.66\times 10^9 \text{ Nmm}^2$$

$$B_B = \sum_{i=1}^{n} E_i A_i z_i^2 = 2.767\times 10^{12} \text{ Nmm}^2$$

$$EI_{eff} = B_A + B_B = (0.08 + 2.767)\times 10^{12} = 2.86\times 10^{12} \text{ Nmm}^2$$

To calculate the distance a:

$$a = t_{total} - \frac{t_1}{2} - \frac{t_n}{2} = 175 - \frac{35}{2} - \frac{35}{2} = 140 \text{ mm}$$

$$GA_{eff} = \frac{140^2}{\left[\left(\frac{35}{2\times 500\times 1000}\right) + \left(\frac{35}{50\times 1000} + \frac{35}{500\times 1000} + \frac{35}{50\times 1000}\right) + \left(\frac{35}{2\times 500\times 1000}\right)\right]} = 12.7\times 10^6 N$$

A simplified method that is used in the CLT Handbook USA is then used to calculate the section modulus:

$$Z_{SAM} = \frac{2\times 2.86\times 10^{12}}{8000\times 175} = 4.09\times 10^6 \text{ mm}^3$$

$$M_d = 0.95\times 0.57\times 1.33\times 12.6\times 4.09\times 10^6 = 37.1 \text{ kNm}$$

B.3 Serviceability

B.3.1 Short-term deflection

The deflection is straight forward to calculate once the effective stiffness has been calculated. The effective stiffness for the Gamma, Composite and Shear analogy method are detailed in B.2. This calculation does not consider load combinations for design. When calculating the deflections for a design scenario the appropriate load case factors in accordance with AS 1170 and AS 1720 should be considered. Only the self-weight of the floor is considered in this example. A comparison of the deflection results for short-term and long-term deflection are included in Table 23.

The density is 460 kg/m^3. Therefore:

w = 460 x 5.8 x 1 =

$$w = 460\times 0.175\times 1\times \frac{9.81}{1000} = 0.8\ k\,N/m$$

Gamma method

$$\Delta_{mid,\gamma} = \frac{5wl^4}{384(EI)_{eff}} = \frac{5\times 0.8\times 5800^4}{384\times 2.68\times 10^{12}} = 4.4 \text{ mm}$$

Composite method

$$\Delta_{mid,k} = \frac{5wl^4}{384k_1 E_0 I_{gross}} = \frac{5\times 0.8\times 5800^4}{384\times 2.85\times 10^{12}} = 4.14 \text{ mm}$$

Shear analogy method

$$\Delta_{mid,SAM} = \frac{5wl^4}{384(EI)_{eff}} + \frac{wl^2k}{8(GA)_{eff}} = \frac{5\times 0.8\times 5800^4}{384\times 2.86\times 10^{12}} + \frac{0.8\times 5800^2\times 1.2}{8\times 12.7\times 10^6} = (4.12 + 0.32) = 4.44 \text{ mm}$$

Table 23: Deflection results comparison of a 2 m wide, 5.8 m long CLT panel.

Property	Gamma	Composite	Shear Analogy
EI_{eff} Nmm^2/m	2.68×10^{12}	2.85×10^{12}	2.86×10^{12}
GA_{eff} N	-	-	12.7×10^{6}
Δ_s mm	4.4	4.14	4.44
Δ_l mm	9.24	8.7	9.32

B.3.2 Long-term deflection

A service class 2 structure is considered and therefore a k_{def} = 1.1 is used to calculate the long-term deflections.

Gamma method

$$\Delta_{mid,\gamma} = \frac{5wl^4}{384(EI)_{eff}} = \frac{5\times0.8\times5800^4}{384\times2.68\times10^{12}/(1+1.1)} = 9.24 \text{ mm}$$

Composite method

$$\Delta_{mid,k} = \frac{5wl^4}{384k_1E_0I_{gross}} = \frac{5\times0.8\times5800^4}{384\times2.85\times10^{12}/(1+1.1)} = 8.70 \text{ mm}$$

Shear analogy method

$$\Delta_{mid,SAM} = \frac{5wl^4}{384(EI)_{eff}} + \frac{wl^2k}{8(GA)_{eff}} = \frac{5\times0.8\times5800^4}{384\times2.86\times10^{12}/(1+1.1)} + \frac{0.8\times5800^2\times1.2}{8\times12.7\times10^6/(1+1.1)} = (8.65+0.67) = 9.32 \text{ mm}$$

B.3.3 Vibration design

Worked solution for the floor is covered in section 4.5.